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PORTAL KNEE CONNECTIONS

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Wood Processors & Manufacturers Association of New Zealand

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1.0 Introduction

1.1 About this guide

Connections between timber elements are often perceived as the trickiest part of design of a timber structure, however done well, they can be economic, efficient to build and visually attractive. This guide focuses on the detailing and layout of a range of common moment connections used between a column and rafter or beam for laminated veneer lumber (LVL) and glue laminated timber (Glulam). This connection is referred to as a knee joint as commonly used for portal frames. The same principle for these connections can be applied to other locations such as the apex joint of the portal.

The types of connections presented have been used in service and proven by manufacture, design and installation. This guide is not intended to be comprehensive as other connection types and timber sizes are available.

1.2 Basis of Design

The designs in this guide are based on NZS3603 and parts of NZS AS1720 (in draft at the time of publication) which is consistent with the design philosophy in the EXPAN guides "Timber Portal Frames" and the TIF Timber Design Guide edited by Andy Buchanan. Some guidance has been taken from proprietary fasteners that have their characteristics values European Technical Approvals (ETA) and we have adopted this in conjunction with New Zealand timber codes to develop a design solution.

It is the responsibility of the designer to check if there are any updates to NZS AS 1720.1

Design of all structural timber shall be in accordance with regulations as advised by the consultants and the following standards:

NZS 3603:1993 – Timber structures standard

AS/NZS 1328:1998 – Glue-laminated structural timber

AS/NZS 2269.0:2008 Plywood-Structural – Part 0: Specifications

AS/NZS 4357:2005 – Structural laminated veneer lumber

NZS 3631:1988 – New Zealand timber grading rules

AS/NZS 1491:1996 – Finger jointed structural timber

AS/NZS 4364:2010 – Adhesives, phenolic and amnioplastic for load bearing timber structures: classification and performance requirements

Eurocode 5: Design of Timber Structures

1.3 Economics and Efficiency of Design

The detailing for the connections chosen in this guide focus on fabrication, buildability and practical installation as well as efficiency of design. Optimal outcomes for timber connections are often a combination of cost effectiveness, speed of construction, visual aesthetics and digital fabrication. Timber has unique properties for connections and typically many small fasteners perform better than larger fasteners.

Table 1 is a guide towards the type of connection that would best suit a project requirements.

Table 1. Pros and Cons of Connection Types

Connection Type	Pros	Cons	Approximate Cost Ratio (assuming Nailed Ply Gusset is 1)
Nailed Ply Gusset	High moment capacity	Time consuming to nail off on site	1.0
	Cost effective	Industrial looking	
Lap Joint	1/2 the number of nails of a ply gusset	Requires blocking between double member	0.3
	No additional gusset material required	Constraint on Overlap area restricts moment capacity	
Screwed Steel Gusset	Higher moment capacity	Labour intensive to drive screws	2.0
		Low fire resistance - if not protected	
Epoxied rods (inc steel knee)	No visible connections - looks clean	Epoxy work should be done in factory	4.0
	Quick to erect on site	Steel knee expensive to fabricate	
		Fire protection maybe required	
Quick Connect	Quick to erect on site	May experience joint rotation	4.0
	High moment capacity	Expensive due to number of components	
WS Dowel and Steel Plate	Looks clean with only dowel heads exposed	Some specialist equipment and training required on site	1.6
	Inherent fire protection		
	Allows for complex joints		

A simple way to make cost savings on timber structures is to specify standard section sizes. Specifying non-standard sizes may achieve optimal structural efficiency and perceived economy but can add up to 20% onto the cost of the structure. Specifying standard sizes is particularly important when designing with LVL. LVL is initially produced in billets or slabs nominally 1200 mm deep and in thicknesses of 45, 63 and 90 mm. Slabs with thickness up to 300mm or more can be produced for special orders by laminating several billets together. Other thicknesses can be made by laminating two 45mm billets together. Readily available section sizes will depend on the timber product specified.

Standard section sizes are cut from the billets with the aim of minimising waste. A designer should attempt to design LVL structures finding usages for any off-cut timber. For example, if a 900 mm deep portal frame section is required, the remaining 300 mm of material cut from the billet is often well utilised as secondary members such as purlins or girts. Furthermore, if the offcuts are standard sizes and not required in the project, then the manufacturer can sell these to other customers, reducing the waste and cost of the project.

Glulam sizes are more flexible as it is relatively simple to add extra laminations to increase the depth of members and structural optimization can be more readily achieved. Attention should still be paid however to using standard widths. Furthermore, where curved glulam members are used lamination thicknesses may have to be decreased to achieve the desired radius of curvature. Smaller radii require thinner timber laminates which add to the gluing and handling costs in manufacture. Experienced glulam manufacturers can advise on the most cost efficient use of material and design aspects.

Fabricators can be called upon to produce custom sections. Examples of work done by fabricators include producing composite sections such as box and deep I-beams, or simply producing custom thicknesses by laminating multiple sections together. Fabricators can add between 20 – 50% of the base price of the material. In some instances this may be a worthwhile investment. For example, box- and deep I-sections often comfortably provide the most efficient solution for long span portals and can offer significant savings on the volume of material required for the frames. However, with regard to cost in general, careful thought should be given to the requirement for fabricators in the supply process.

Timber portal frame structures can often be built using untreated wood products. The open nature of these buildings provides good airflow around the timber members, preventing the build up of moisture. Where treatment is required however there will be an associated cost may be up to 25% of the base price of the material.

Where timber products are supplied through a merchant, a 20-30% increase in price can be expected on the base price of the timber from the manufacturer.

Finally, of course an allowance should be made for the cost of construction. The construction company is likely to add 10% to the cost of materials. The figures given before are only approximate estimates of what contribution to cost each stage of the supply chain can add.

Other considerations should include the onsite fabrication and erection procedures.

The requirement of each stage is of course dependant on the building's design. It is therefore the role of the design engineer to consider the path from the manufacturer to construction and what implications on the overall project cost it may have.

1.4 Treatment and Durability

Wood preservation extends the useful life of timber by modifying its resistance to detrimental agents such as insect attack and fungal decay. The design of timber buildings must aim to prevent the deterioration of timber by providing adequate levels of protection.

Portal frame buildings typically feature open interior spaces, allowing good air flow through the building. Therefore timber portal frames can often be built from untreated timber. Some cases where treatment may be required would include industrial plants where high humidity and temperatures are expected.

It is important to ensure timber members are protected from the weather and moisture during construction. For example, to prevent moisture entering the end grain of columns during construction, it is recommended to provide a moisture barrier at each end of the column.

1.5 Fire Considerations

The performance of timber structures under fire conditions is a common concern of design engineers and clients alike. However, contrary to common belief, large solid timber members perform well under fire conditions.

The charring rate of Radiata Pine (LVL and Glulam) is given in NZS 3603:1993 as 0.65 mm per minute. The residual section of a timber member under a particular fire loading can be found by subtracting a thickness equal to the charring rate multiplied by the fire rating period (in minutes) from all faces of the member which are exposed to the fire.

For most portal frame buildings, the structural elements support non-trafficable roofs and hence are not required to have a fire rating. Where fire ratings are necessary however, timber members can be protected by using many of the same means as used for structural steel members. Where a fire rating is necessary, particular attention should be paid to the joints of the structure. Plywood and steel gussets offer minimal fire resistance, as do proprietary fixing brackets and joist hangers often used to support purlins and girts. Measures should be taken to ensure the adequacy of these connections under fire conditions.

(Above and below) NSW Netball Center using LVL portals and Quick Connect Connections

2.0 Timber Materials

The connections based in this guide are for Glulam and Laminated Veneer Lumber (LVL). These materials are made in large section sizes and long lengths and are two of the most common engineered timber post and beam products available on the market.

2.1 Glulam

2.1.1 Sizes

These standard connection details are based on commonly available straight sections of glulam.

In order to simplify the design process used in this guide's examples, a range of section sizes from 450mm to 900mm have been used.

Glulam Sections

2.1.2 Structural Grades

In New Zealand, glulam is generally produced in three grades, GL8, GL10 and GL12. GL17 is also manufactured in New Zealand, please consult your supplier for availability. GL8 is the most readily grade available and has been adopted for this guide. Table 2 gives the characteristic stresses for glulam grades.

Table 2. Glulam Characteristic Stresses (NZS AS1720.1 Draft for Comment 2020)

Grade	Bending f_b	Shear in beams f_s	Compression parallel to grain f_c	Tension f_t	Modulus of Elasticity E	Modulus of rigidity G
	MPa	MPa	MPa	MPa	MPa	MPa
GL18	50	5.0	50	25	18500	1200
GL17	42	3.7	35	21	16700	1100
GL13	33	3.7	33	16	13300	890
GL12	25	3.7	29	12.5	11500	770
GL10	22	3.7	26	11	10000	670
GL8	19	3.7	24	10	8000	530

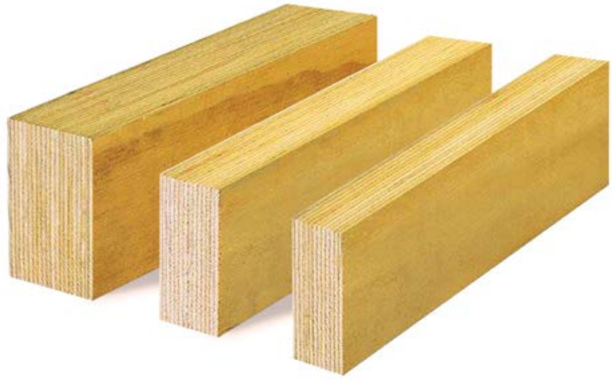
2.2 Laminated Veneer Lumber – LVL

LVL is an engineered timber product manufactured by glue laminating timber veneers usually in a continuous assembly production. The grain orientation of all veneers is normally in the longitudinal direction which gives orthotropic properties similar to sawn timber. The characteristic stress values are higher in LVL as the lower variation in the timber properties.

Perpendicular veneers may be included in the LVL lay-up to provide extra strength in the perpendicular to the normal grain direction and this referred to as crossbanded LVL.

2.2.1 Sizes

LVL section sizes should be selected to make full use of the 1200mm billet. The sizes used in this guide are the common sizes. The thicknesses presented are made up of multiples of 45mm members glued together. Other thickness can be achieved.



LVL Sections

2.2.2 Structural Grades

Standard LVL grades have been proposed in the new Timber Design Standard NZS AS 1720.1 – Timber Structures and these have been published in the draft standard. The published values are generally conservative when compared to manufacturer's data. For the standard connections in this guide LVL13 has been adopted. Refer to Table 3 below.

Table 3. LVL Characteristic Stresses (NZS AS1720.1 Draft for Comment 2020)

				Characteristic values for LVL beam on edge					
LVL grade	Design Density	Characteristic density at 12% MC	Bending f_b	Shear in beams f_v	Compression perpendicular to grain f_{px}	Tension parallel to grain f_t	Compression parallel to grain f_c	Average Modulus of Elasticity E	Modulus of rigidity G
	kg/m3	kg/m3	MPa	MPa	MPa	MPa	MPa	MPa	MPa
LVL16	600	480	50	4.5	10	25	45	16000	800
LVL13	600	480	45	4.0	10	25	38	13200	660
LVL11	570	480	38	3.5	10	16	32	11000	550
LVL10	540	480	35	3.5	10	15	30	10000	500
LVL8	480	480	30	3.5	10	15	30	8000	400

Notes: For beams with depth exceeding 95mm multiply the value for bending (f_b) by $(95/d)^{0.167}$ where d is the depth of the beam.

2.2.3 Joint strength groups

The joint strength groups for LVL vary depending on the type of fastener and there are also differences between the LVL manufacturers and products. In the following Table 4 gives conservative joint groups for common fasteners. These have been adopted for this design guide.

Table 4. LVL13 Joint Group Classification

LVL Grade	Nails and screws in lateral load	Nails and screws in withdrawal	Self drilling (Type 17) screws in lateral load	Self drilling (Type 17) screws in withdrawal	Bolts or coach screws in lateral load into the face
LVL 13	J5	J5	J4	J4	J3

2.3 Plywood

2.3.1 Size

Normal plywood sheet sizes are 1.2 m x 2.4 m or 1.2 m x 2.7 m. Regularly available thicknesses are 7, 9, 12, 15, 17, 19, 21 and 25 mm.

The number of plys (veneers) making up the total thickness of the plywood sheet will vary depending on the manufacturer.

2.3.2 Grades

In New Zealand, structural plywood is readily available in two stress grades, F8 and F11. Grade F17 plywood can sometimes be obtained as a special order. A range of higher grades are commonly available in Australia. Table 5 gives the characteristic stresses as given in NZS3603.

Table 5. Characteristic Stresses For Structural Plywood

Stress grade	Bending f_{pb}	Tension f_{pt}	Panel Shear f_{ps}	Rolling Shear f_{pr}	Compression in the plane of the sheet f_{pc}	Compression normal to the plane of the sheet f_{pp}	Modulus of elasticity E	Modulus of rigidity G
F22	57.6	34.6	6.0	2.4	43.2	20.4	16000	800
F17	44.5	26.7	6.0	2.4	33.4	17.3	14000	700
F14	36.7	22.0	5.4	2.2	27.5	13.6	12000	625
F11	28.8	17.3	4.7	1.9	21.6	10.7	10500	525
F8	22.5	13.5	4.2	1.7	16.9	8.6	9100	455

2.4 Cross-Banded LVL

An innovation in the development of LVL is cross-banded LVL. This is produced by laying veneers across the line during the LVL lay-up process. The cross bands are similar to plywood although cross bended LVL generally has only 2 cross veneers while plywood has every alternate layer.

The cross veneers assist in controlling cupping in deeper sections while providing extra strength in the perpendicular to grain direction, although the strength and stiffness in the direction of the grain are reduced.

Cross-banded LVL can be made in greater lengths and thicknesses than standard plywood. The length of a piece of cross-band LVL is only limited by the maximum length of an LVL billet, which is typically 18 m. This makes cross-band LVL an excellent material for webs of box beams, as the requirement for web splices is generally removed. It also allows for a tighter nailing pattern than regular LVL and thus has been used effectively to form moment resisting knee and apex joints in LVL portal frames. When used for composite beams, cross-banded LVL will provide a much greater contribution to the members overall strength and stiffness than plywood. Refer to manufacturers for characteristic stresses and readily available sizes.

3.0 Fasteners Types for Timber

Portal frame buildings encompass the usage of most regular timber fasteners. Nailed gusset connections provide the most common form of moment resisting joint used in timber portals. Some designers prefer to use screws for these joints instead of nails. Both nails and screws are regularly used in conjunction with proprietary fixing brackets to support roof and wall elements. Screws are more expensive to install than nails but create connections with significantly higher capacity. All fasteners should be specified with a minimum of electroplated coating. Exposure to the weather during construction may result in some rusting of uncoated fasteners and subsequent discolouration of the timber members.

Bolts are generally used to provide simple connections at the base of columns and connections between steel elements. New fasteners, such as self-drilling dowels or screws in Figure 1 are now available enhancing the range of connection systems available.

Consideration of compatibility between fasteners and treated timber shall be considered especially in humid or wet environments.

Table 6. NZS3603 Characteristic strengths (N) for nails in side grain single shear (dry timber)

Timber group	Nail shank diameter (mm)													
	2.0	2.24	2.50	2.80	2.87	3.15	3.33	3.55	3.75	4.00	4.50	5.00	5.30	6.00
J5	268	331	407	504	526	631	695	790	868	990	1240	1510	1690	2130
J4	391	476	577	703	733	863	951	1060	1165	1310	1610	1930	2140	2660
J3	550	671	812	990	1030	1220	1345	1500	1650	1840	2270	2720	3010	3740
J2	680	824	993	1200	1250	1470	1620	1800	1980	2200	2690	3210	3540	4370
J1	743	908	1100	1350	1410	1660	1830	2060	2260	2540	3130	3770	4190	5220

Table 7. NZS3603 Characteristic strength (N) for wood screws in side grain single shear (dry timber)

Timber group	Minimum screw shank diameter (mm)								
	2.74	3.10	3.45	3.81	4.17	4.52	4.88	5.56	6.30
	Screw gauge number								
	4	5	6	7	8	9	10	12	14
J5	554	700	854	1025	1229	1429	1652	2140	2663
J4	775	960	1155	1371	1614	1855	2118	2692	3303
J3	1091	1356	1634	1938	2270	2615	2985	3786	4663
J2	1324	1635	1964	2323	2705	3098	3526	4439	5429
J1	1486	1846	2235	2660	3132	3606	4133	5276	6503

NOTE – Maximum screw shank diameter = above mentioned shank diameter + 0.13mm

Figure 1. Modern SBD self-drilling screw

4.0 Connections

4.1. Introduction

This section outlines the theory and design process for several types of connections for a typical knee joint for portal frames as commonly used for industrial building. The design process can easily be adapted for other joint configurations.

The drawings provided illustrate typical joints along with their moment capacities for a variety of common LVL and Glulam sizes. These are intended as a starting point for design and should be verified by the design engineer.

4.2. Ductility Factors For Seismic Design

The following ductility factors, in Table 8, are considered appropriate for calculating seismic loads in accordance with AS/NZS1170 loads. Higher ductility factors may be considered by engineers experienced in ductile design and detailing of connections.

Table 8. Recommended Ductility Factors For Different Connection Types

Connection type	Ductility Factor - μ
Nailed Gusset - connections	1.25 - 1.5
Screwed Gusset - connections	1.25
Epoxy Rod into Steel bracket	1.25
Epoxy Rod into timber only	1.0
Glued connections	1.0
Quick Connect	1.0
WS dowels in steel plates	1.25 - 1.5

Plywood Gusset Portal

4.3 Moment Knee Connections – Nailed Ply Gusset (MKCNPG)

4.3.1 Introduction

Nailed gusset joints for timber portal frame buildings provide a cost-effective connection. Due to their appearance, they are less favoured for high profile buildings.

These joints are constructed using plywood or cross-band LVL gussets nailed to each side of the members. Figure 2 shows four different types of gusset joint layouts. The internal haunch is the most common form.

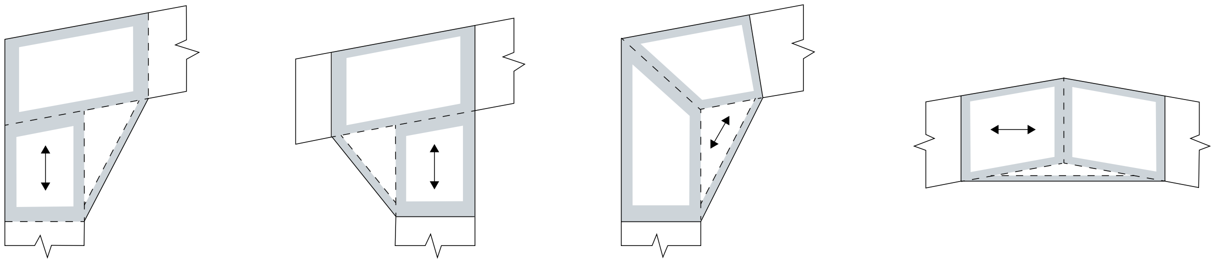


Figure 2. Different Configurations of Plywood Gusset Joints

The standard sizes of plywood sheets available will limit the size of the gusset. Where a double layer of ply is used, gluing the sheets together with builders adhesive, reduces nail slip between the sheets, otherwise the nail capacity is reduced.

A steel plate can be used for the gusset as this will develop the full flexural strength of the timber members without the need for a haunch. However, the plates must be pre-drilled for the nails which are then be driven by hand or by specialist nail gun which is labour-intensive

Where fire rating is required measures must be taken to protect the joint. Common methods of providing fire protection include covering vulnerable elements with either adequate levels of sacrificial timber or non-combustible overlays.

4.3.2 Design Actions

The design actions are obtained from a suitable structural analysis. Although bending moment dominates the connection design, the effect of axial and shear forces through the joint must still be considered.

The critical section of the gusset is taken as a horizontal line where the column centreline intersects the rafter soffit. While the design bending moment is the same for both the rafter and column sections, the axial and shear forces are taken from the column design actions.

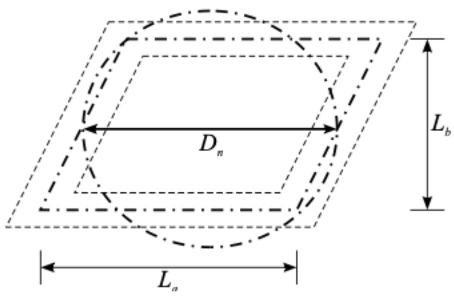


Figure 3. Nail Group Layout

To check the strength of the gusset the design bending moment, axial and shear force should be taken at the point of intersection between the column centreline and the rafter soffit. Design actions for the nail rings should be taken at the centroid of the nail group, see Figure 3.

4.3.3 Gusset Design

The gusset must be capable of transferring axial forces, bending moments and shear forces across the joint.

Figure 4 shows the stresses that result in gusset joints under bending moment as suggested by Batchelar (1984). Knee and apex gussets have different stress distributions and critical sections resulting in differences in their design.

For knee joints there are two commonly adopted methods for establishing the moment capacity of the gusset.

Method 1. Batchelar Method

Batchelar modelled the bending stresses in the gusset using a bilinear distribution. A constant stress distribution is assumed in the gusset outside of the line of the portal column member, while stress is assumed to vary linearly along the line of intersection of the portal rafter and column members (Batchelar, 1984).

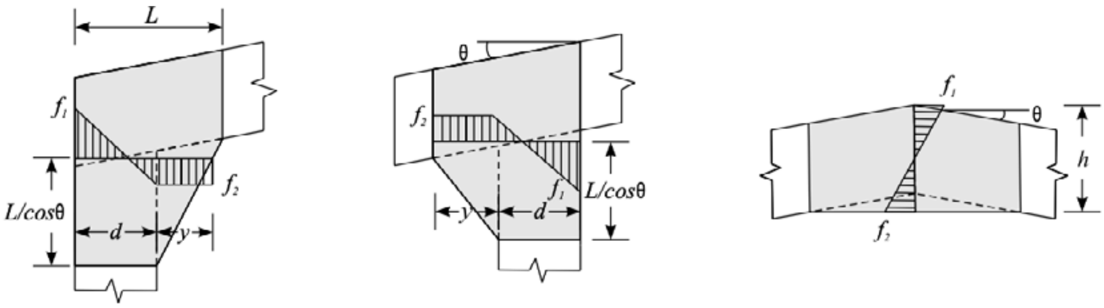


Figure 4. Assumed Distributions In Plywood Gusset Joints (Buchanan, 2007)

The critical section is taken as a horizontal line through the point of intersection of the portal column centreline and rafter soffit. Stresses at the critical section are given by:

$$f_1 = \frac{24Mk(1-k)}{td^2(4k-1)}$$

and

$$f_2 = \frac{f_1(1-k)}{k}$$

where

$$k = \frac{(y+\frac{d}{2})}{(y+d)}$$

$$y = \frac{L-d}{1+(1-\frac{d}{2L})\sin\theta}$$

and

M = design bending moment in joint

t = effective plywood thickness

d = depth of portal column

ϕ = slope of member

f_1 is typically the greater of the two stresses. By rearranging and adding strength reduction modification factors from NZS 3603:1993 an expression for the moment capacity of the plywood gusset can be obtained:

$$\phi M_n = \phi k_1 f_b \frac{td^2(4k-1)}{24k(1-k)}$$

where

ϕ = strength reduction factor

= 0.9 for plywood and LVL

k_1 = load duration factor

f_b = characteristic bending strength of the gusset material

Equation above for ϕM_n is not given in NZS 3603:1993 but has been adapted from the research of Batchelar (1984) and applies specifically to the bending capacity of knee gusset joints. The moment capacity of the gusset is based on the depth of the portal frame column, not the depth of plywood gusset.

Method 2. Hutchings Method

Hutchings modelled the bending stresses in plywood gussets using a linear distribution. Using this method the moment capacity of the gusset can be obtained simply by applying the equation for the nominal in plane bending strength of plywood from NZS 3603:1993.

This model has been adopted for the design of the joint capacities in this guide.

$$\phi M_n = \phi k_1 k_8 k_{14} k_{15} k_{24} f_{pb} \frac{(t_e d_{cs}^{\ 2})}{6}$$

In this instance the moment capacity is dependent on the depth of the critical section through the gusset which is given by:

$$d_{cs} = y+d$$

Where d is the depth of the portal column member and y is given above.

The critical section depth can then be applied in conjunction with the equations from NZS 3603:1993 to obtain the shear and axial force capacities for the plywood gusset.

$$\phi N_{nc} = \phi k_1 k_8 k_{14} k_{15} f_{pc} t_e d_{cs}$$

$$\phi N_{nt} = \phi k_1 k_{14} k_{15} f_{pt} t_e d_{cs}$$

$$\phi V_{ni} = \phi k_1 k_8 k_{14} k_{15} k_{18} f_{ps}\ {}^2\!/_3\ t d_{cs}$$

where

ϕ = strength reduction factor, 0.9 for plywood and LVL

k_1 = load duration factor

k_8 = stability factor 1.0 for gussets nailed on all sides

k_{14} = moisture content factor- normally 1.0 for interior use

k_{15} = face grain orientation factor, normally 1.0 unless plywood is placed on an angle

k_{18} = panel shear framing support factor, normally 1.0 as plywood is nailed around all edges

k_{24} = size factor for glulam member = 1.0 when using the stresses for glulam grades

$k_{24,LVL}$ = size factor for LVL member depth > 95 mm

$$\left(\frac{95}{d}\right)^{0.167}$$

f_{pc} = plywood characteristic stress in compression

f_{ps} = plywood characteristic shear strength

f_{pt} = plywood characteristic tensile strength

t_e = effective gusset thickness

t = total gusset thickness

d_{cs} = gusset critical section depth

The equations are obtained from the plywood chapter of the timber structures standard and are also applicable to cross-band LVL. The effective thickness depends the number of veneers orientated parallel and perpendicular to the critical section.

A stiffener should be included along the free edge of gussets to provide stability against buckling.

Apex Gussets

The difference in geometry and stress distribution between apex and knee joints must be taken account. For the design of apex gussets a linear stress distribution is observed (see Figure 4) with the maximum stress being equal at the top and bottom.

Bending stresses in apex gussets are given by:

$$f_1 = f_2 = \frac{M}{Z}$$

By rearranging and applying modification factors from NZS 3603:1993 the moment capacity of a ridge gusset is given by:

$$\phi M_n = \phi k_1 f_{pb} Z$$

The critical section occurs at the interface between the two rafter members. The effective depth of the gusset at the critical section is given by:

$$d_{cs} = h \cos\theta$$

Where h is the total depth of the gusset.

The section modulus Z of the ridge gusset in bending is then given by:

$$Z = \frac{(th^2 \cos^2\theta)}{6}$$

Having calculated the moment capacity of the ridge gusset, shear and axial capacities are then calculated using the same equations as with knee joints.

Combined Actions Checks

Combined action checks for bending, shear and axial load are carried out using Equations 6.17 and 6.18 of NZS 3603:1993:

$$\left(\frac{N_c^*}{\phi N_{nc}}\right) + \left(\frac{M_i^*}{\phi M_{ni}}\right)^2 + \left(\frac{V_i^*}{\phi V_{ni}}\right) \leq 1.0$$

$$\left(\frac{N_i^*}{\phi N_{nt}}\right) + \left(\frac{M_i^*}{\phi M_{ni}}\right)^2 + \left(\frac{V_i^*}{\phi V_{ni}}\right) \leq 1.0$$

Nail Group Design

The design bending moments, axial and shear forces are all transferred through the nails between the rafters and columns.

The majority of load carried by the nail group is the result of the bending moment carried by the joint. The moment capacity of the nail group is calculated first by adopting a rivet group analogy. The remaining lateral capacity of the joint is then checked against the vectorial sum of axial and shear forces.

The nail pattern adopted shown in Figure 5 can be used for both along and across the grain and is suitable for nails up to 3.3mm diameter. This pattern minimises the chance of incorrect installation and allows a simple nailing template to be used by the fabricator. Flat head nails must be used and care should be taken not to overdrive the nails when using a nail gun.

Check with NZS3603 for nail spacing if using a timber species other than Radiata Pine.

The calculations for the capacity of the nail group can be labour intensive for large nail groups. Such calculations can be simplified by adopting rectangular or parallelogram shaped nail rings with two axes of symmetry to determine the average radius. Drawing the joint in CAD to measure the axes graphically is also a viable method.

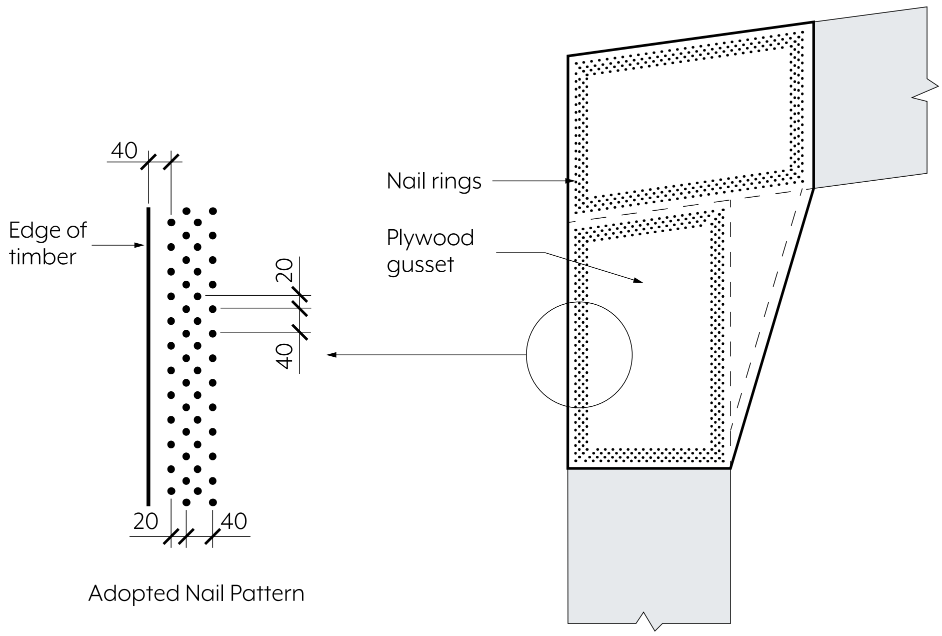


Figure 5. Nail Pattern

Table 9. Minimum spacing for nails in Radiata Pine

Distance	Minimum spacing
End distance	$12d_a$
Edge distance	$5d_a$
Between nails along grain	$10d_a$
Between nails across grain	$5d_a$

Where d_a = nail diameter

Moment Capacity of a Nail Group

The capacity of a nail group subject to in-plane bending moments must satisfy:

$$M^* \leq \phi Q_n$$

$$\phi Q_n = \phi \ \frac{k}{r_{max} \sum_{i=n}^{i=1} r_i^2}$$

And for nail groups under direct lateral loads (axial and shear forces):

$$\phi Q_n = \phi n k Q_k$$

where

ϕ = strength reduction factor

0.8 for nails in lateral loading

n = number of nails in the connection

k = product of the following modification factors

- Load duration factor k_1
- 1.25 Steel side plate, thickness < 3.0 mm thickness
- 1.5 Steel side plate, thickness ≥ 3.0 mm thickness
- 1.4 Plywood, or cross band LVL with flat head nails.
- 1.3 for connections containing 50 or more nails.

r_i = distance from the i^{th} nail to the centroid of the nail group

r_{max} = the maximum value of r_i

Q_k = characteristic strength of a single nail in shear

It has been found that a simplified calculation for the nail group moment using the average radius of the nail rings will give a close enough result for most design purposes.

$$\phi M_n = \phi n k r_{av} Q_k$$

Where

r_{av} = the average radius of the all nail rings

with other terms as defined above.

Remember that there will normally be a gusset each side of the joint when calculating the overall joint capacity.

After calculating the moment capacity of the joint, the remaining capacity should be checked against the design axial and shear forces.

$$N^*_{axial/shear} \leq \phi N_{axial/shear}$$

The vectorial sum of the axial and shear forces in the joint is given by:

$$N^*_{axial/shear} = max \begin{cases} \sqrt{(N_c^*)^2 + (V^*)^2} \\ \sqrt{(N_t^*)^2 + (V^*)^2} \end{cases}$$

The capacity of the joint to carry these forces over and above the bending moment in the joint is then:

$$\phi N_{axial/shear} = \left(1 - \frac{M^*}{\phi M_n}\right) \times \phi Q_n \times n$$

4.1.3 Joint Rotation in Nailed Gusset Joints

Nail slip can cause significant joint rotation which will increase overall deflections in the portal frame.

The slip in a nail is calculated using:

$$\delta = 0.8 k_{37} \left(\frac{P}{Q_n}\right)^2$$

where

δ = nail slip (mm)

k_{37} = load duration factor for fastener slip (Table 10)

P = applied nail load

Q_n = nominal strength per nail with short term loading ($k_1 = 1$)

Table 10. Load duration factor k_{37} for fastener slip for dry timber (From table E1 NZS 3603:1993)

Duration of Load	Factor k_{37}
More than 6 months	5
2 weeks to 6 months	2
5 minutes to 2 weeks	1.5
Less than 5 minutes	1

The overall structure deformation can be determined from the joint rotation after the nail slip has been calculated.

The joint rotation (rad) in radians is calculated by dividing the joint deformation above by the average radius or distance from the centre of the connection within the joint. For nailed gusset joints, or other joints where the offset distance varies for each fastener, a quick approximation is to consider the joint group to be a circular band with a notional centreline diameter as shown in Figure 4.

The joint rotation in radians is then given by:

$$\theta = \frac{rad}{0.5D_n}$$

where

$$D_n = 0.75\sqrt{(L_a^{\,2} + L_b^{\,2})}$$

The average fastener offset is then $0.5D_n$.

L_a and L_b are taken from Figure 3.

The final vertical deflection at the apex due to rotation at the knee is calculated by multiplying the joint rotation by half of the portal frame span:

$$\Delta_{apex} = \theta \times \frac{L}{2}$$

The same approach is applied for fully fixed frames, starting from the base connections and working through to the apex.

Use of Screws

The design process for screws is essentially the same as for nails. Characteristic strengths and spacings for screws are provided in NZS3603:1993. The embedment into the second layer should be at least 7 times the fastener diameter. Screws can exhibit a brittle failure, so caution is advised if a ductility factor greater than 1.25 is used for seismic design. See standard details in 6.0 Appendix for reference.

4.3.4 Table of Capacities

The following Tables 11 and 12 summarise nailed gusset joint moment capacities for Glulam and LVL members. Capacities are for a load duration factor k_1 = 1.0.

Table 11. Glulam GL8 Moment Connection Capacities - Nailed Ply Gusset (MKCNPG)

Member Size D x B (mm)	Member capacity f_b = 19MPa ØMn (kNm)	Gusset thickness /type (each side)	Gusset length (mm)	Nail diameter (mm)	Number of nail rings	Gusset moment capacity ØMn (kNm)	Nailed joint bending capacity ØMn (kNm)
450 x 90	46	1 layer 25mm F8 ply	750	2.8	3	48	57
540 x 90	66	1 layer 25mm F8 ply	900	2.8	3	69	92
630 x 90	90	1 layer 25mm F8 ply	1000	2.8	3	86	122
720 x 90	118	1 layer 25mm F8 ply	1200	2.8	3	123	184
720 x 135	177	2 layers 19mm F8 ply	1200	2.8	3	192	184
810 x 90	150	2 layers 17mm F8 ply	1200	2.8	3	180	203
810 x 135	224	2 layers 25mm F8 ply	1200	3.15	3	217	255
900 x 90	185	2 layers 19mm F8 ply	1200	2.8	3	200	224
900 x 135	277	2 layers 25mm F8 ply	1200	3.15	3	256	281

Table 12. LVL 13 Moment Connection Capacities - Nailed Ply Gusset (MKCNPG)

Member Size D x B (mm)	Member capacity f_b = 45MPa ØMn (kNm)	Gusset thickness /type (each side)	Gusset length (mm)	Nail diameter (mm)	Number of nail rings	Gusset moment capacity ØMn K_l = 1 (kNm)	Nailed joint capacity ØMn K_l = 1 (kNm)
450 x 90	95	2 layers 21mm F8 ply	750	3.15	3	83	71
600 x 90	161	2 layers 21mm F8 ply	900	3.15	3	125	125
600 x 135	241	2 layers 25mm F8 ply	1000	3.15	4	170	181
800 x 90	272	2 layers 25mm F8 ply	1200	3.15	3	250	251
800 x 135	409	2 layers 25mm F11 ply	1200	3.15	4	320	314
900 x 90	338	2 layers 25mm F11 ply	1200	3.15	3	328	281
900 x 135	507	2 layers 25mm F11 ply	1200	3.15	4	328	351
1200 x 90	573	2 layers 25mm F11 ply	1200	3.15	3	345	380
1200 x 135	859	1 layer 45mm LVL 13	1200	3.15	4	573	479
1200 x 180	1146	1 layer 45mm LVL 13	1200	3.15	5	573	566

Marshlands School using LVL portals and LVL gussets.

4.4. Moment Knee Connections – Lap Joint (MKCLJ)

Figure 6 Lap Joint

A variation on the nailed gusset is the lapped nailed joint. These joints feature two rafter members lapped each side of a single column member or vice versa. The nail rings penetrate though the outside members from both sides into the central member.

The outer members are typically only 45 mm thick for the nails to achieve the required penetration into the central member. The load transfer mechanism for a nailed lapped joint is essentially the same as with a nailed gusset joint but with a reduced area for the nail groups. Typically, a 75mm or 90mm length nail will achieve the required penetration into the second member.

Bending moments, axial and shear forces are transferred across the joint by means of the nails acting in shear. Unlike gusset joints, a lapped joint has only one nail group on each side of the joint, reducing the labour required.

The design must consider the stability of the slender outside members. Normally, the maximum recommended depth to breadth ratio of an LVL member is 10. At only 45 mm thick, the outer LVL member will likely be well in excess of 450 mm deep so stability calculations are required. Blocking between the members can be added to improve resistance to buckling.

4.4.1. Table of Capacities

The following tables summarise the joint moment capacities for Glulam and LVL lapped joints. The nail spacing adopted is as for the gusset connections. The load duration factor k_1 = 1 is used for the bending capacities given in Table 13 & 14.

Table 13. Glulam GL8 Moment Connection Capacities - Lapped Joint (MKCLJ)

Glulam Member Size	Member capacity f_b = 19MPa ØMn (kNm)	Nail Diameter (mm)	Number of nail rings	Joint Moment Capacity for k_1 = 1.0 ØMn (kNm)
450 x 90	46	3.15	3	32
540 x 90	66	3.15	3	52
630 x 90	90	3.15	3	79
720 x 90	118	3.15	3	111
810 x 90	150	3.15	3	150
900 x 90	185	3.15	3	194

Table 14. LVL 13 Moment Connection Capacities - Lapped Joint (MKCLJ)

LVL Member Size	Member capacity f_b = 45MPa ØMn (kNm)	Nail Diameter (mm)	Number of nail rings	Joint Moment Capacity for k_1 = 1.0 ØMn (kNm)
450 x 90	95	3.15	4	34
600 x 90	161	3.15	4	82
800 x 90	272	3.15	4	178
900 x 90	338	3.15	4	240
1200 x 90	573	3.15	4	479

4.5 Moment Knee Connections – Epoxy Rod (MKCER)

Introduction

Epoxy grouted steel rod joints are suited to both glulam and LVL portal frames where the frame size is governed by deflection rather than strength.

When a joint in LVL is subject to an opening moment and the rod is installed perpendicular to grain testing carried out found that there is difficulty developing sufficient strength in the connection due to premature fracture of the beam. Opening moments reflect the loading scenario in portal frame knee joints under wind uplift conditions or reversal in seismic loads. Hence, a steel knee arrangement with the rods installed parallel to grain only is recommended for epoxy dowel rod (threaded rod) connections in timber portal frames.

While both deformed reinforcing rods and fully threaded steel rods can be used, G8.8 threaded rods are recommended. These provide a greater embedment strength and will suit the installation to the steel knee. Also parallel compression stresses can be used in all cases which are higher than perpendicular.

Where seismic loads govern, the steel knee can be designed as the ductile element as often the wood fracture will govern the strength of the rod in timber.

Tests have shown these joints can exhibit deformation due to joint rotation over long periods of time. Currently there is no general design method to account for this. They also should not be employed in environments with a highly variable moisture contents as micro-cracks can develop leading to longitudinal splits in the members.

Epoxy Glues

A low viscosity epoxy should be used in order to ensure that it flows around the deformed or threaded rods, providing complete encasement. High viscosity epoxies may cause premature pull out failures.

The following types of epoxy are recommended:

- Araldite 2005 (Nuplex construction products)
- Araldite K-80 (Nuplex construction products)
- West System ADR310/ADH26 (Adhesive Technologies Ltd)
- West System Z105/Z205 or Z105/Z206 (Adhesive Technologies Ltd)
- East 221 epoxy (Polymer Developments)

The East 221 epoxy has demonstrated good performance in structures for periods of over 20 years. The West System ARD310 resin was used in the Sydney 2000 Olympic Games Project.

Some epoxies may be prone to softening when heated and therefore should not be used where fire resistance is required.

Loctite CR421 Purbond (two component polyurethane) is used in Europe for gluing steel rods inside timber with successful results. This maybe considered in New Zealand – refer to 5.0 Further Reading.

4.5.1. Geometric Recommendations

The geometry of epoxy rod joints has a significant effect on the strength of the connection. Also parallel compression stresses can be used in all cases which are higher than perpendicular stresses. Shear across the connections is generally not a concern, however where a connection has higher shear forces, there is guidance in NZS AS1720 on shear transfer perpendicular to grain. Unless there is further evidence on this, a separate shear plate or shear key should be used for shear transfer.

The following design sequence has been adapted from the recommendations of Batchelar (2006), Buchanan & Moss (1999) and Milner & Crozier (2000). The design process is an optimisation approach. The steps outlined below may need to be repeated in to achieve a satisfactory solution.

Table 15. Geometric Requirements for Epoxy Grouted Rods

Geometric constraint		Notes
Bar diameter	10 - 24mm	Best structural performance is achieved by using a larger number of small rods rather than fewer rods of large diameter.
Hole diameter	$1.25d_b$	Larger holes provide greater strength. Although hole diameters should not exceed $1.5d_b$
Embedment length	$20d_b$	Failure load is proportional to embedment length. Beyond $20d_b$ however there is little increase in strength.
Edge distance	$2.5d_b$	$2.5d_b$ is recommended for joints carrying axial and shear forces. An absolute minimum value of $1.5d_b$ may be used in the absence of shear force.
Spacing between bars in groups	$2d_b$ minimum	Closely spaced bars should have different embedment lengths, staggered by at least 75 mm.

4.5.2 Design Actions

From structural analysis of the portal frame, obtain the design actions at the joint, M^*, N^* and V^*.

Estimate required Area of Steel

A preliminary estimate for the area of steel required can be made by assuming a lever arm of $0.75d$. This gives the tension force N_t^* that the steel rods are required to carry. The steel rods must be checked again for yielding once the rod group layout has been finalised.

$$N_t^* = \frac{M^*}{0.75d}$$

$$N_t^* \leq (\phi Q_n)_{steel}$$

$$(\phi Q_n)_{steel} = \phi_{steel} n A_s f_y$$

where

ϕ_{steel} = 0.8 for bolts and rods in tension (from NZS 3404)

n = number of steel rods acting in tension

A_s = area of a single steel rod

f_y = characteristic yield strength of a steel rod

By rearranging the equation above the required area of steel can be determined.

Tension and Compression Forces in Joint

Batchelar (2006) recommended a mechanics based method for analysing epoxy dowel connections. The joint is treated much like a reinforced concrete beam, with the steel rods carrying the tensile force and the timber carrying the compression force. This is shown in Figure 7.

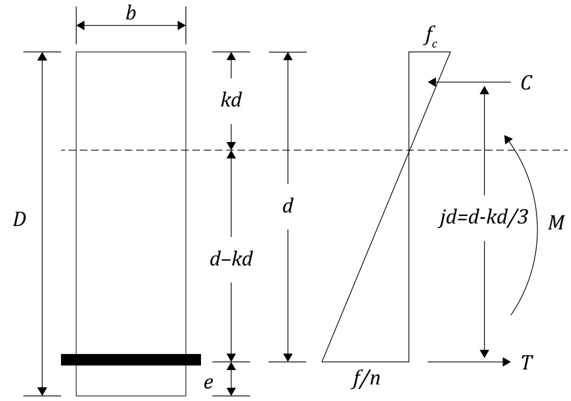


Figure 7. Analysis of an Epoxy Rod Joint

The compression force is given by:

$$C = 0.5f_c bkd$$

and the tension force carried by the steel rods:

$$T = A_s f_y$$

The neutral axis lies at the centroid of the section's transformed area, a depth kd from the compression edge of the member. By taking moments about the neutral axis a quadratic equation can be formed and solved for kd (Batchelar, 2006).

$$b\frac{(kd)^2}{2} = nA_s\,(d–kd)$$

Where n is the modular ratio between the steel and timber elements and is given by:

$$n = \frac{E_{steel}}{E_{timber}}$$

If not using a steel knee joint, the compression bearing surface is loaded perpendicular to the grain. Therefore the E_{timber} value used should be a perpendicular to grain value E_{perp}. E_{perp} is expected to be between 1/20 to 1/30 of $E_{parallel}$.

Once kd has been found, jd can be calculated and the tension and compression forces can be found:

$$jd = d - \frac{kd}{3}$$

$$T = C = \frac{M}{jd}$$

Compression Stress in Timber

The compression force in the joint is carried by the timber. The maximum compression stress (f_c) in the timber can be found by re-arranging the equation above. The following condition must then be satisfied:

$$f_c^* \leq \phi k_1 f_c'$$

Where the compression force is perpendicular to grain, the characteristic perpendicular to grain value f_p must be used instead of f_c. This can be a demanding design criterion. For glulam made from Radiata Pine a value of f_p = 4.5 MPa should be adopted in design.

However, with the use of the steel knee, the rods will be clamped to the steel by the nuts each side which will form a stiff connection relative to the timber.

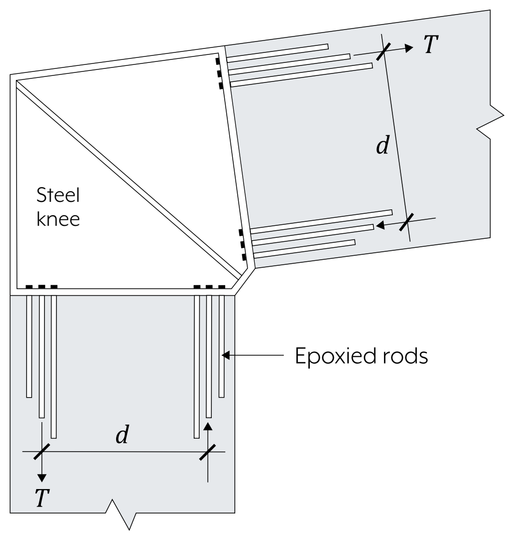


Figure 8. Simplified Moment Couple

Therefore, the centre of compression will tend to be close to the centre of the rod group.

Simplified Method

A simplified method of calculating the moment capacity is to use the centreline of the rod groups as the lever arm distance. With the moment capacity being the total calculated tension capacity of the rods multiplied by the lever arm, as shown in Figure 8.

Wood Fracture at Tips of Steel Rods

Using the tension force calculated above, check for wood fracture at the tips of the embedded steel rods.

$$T \leq (\phi Q_n)_{wood}$$

where

$$(\phi Q_n)_{wood} = \phi_{conn} k_1 A_w f_t$$

and

ϕ_{conn} = 0.7 for wood fasteners other than nails or bolts (refer NZS 3603:1993, clause 2.5)

k_1 = load duration factor (refer NZS 3603:1993, table 2.4)

A_w = net area of wood cross section (excludes holes drilled for steel rods). Refer to Figure 9.

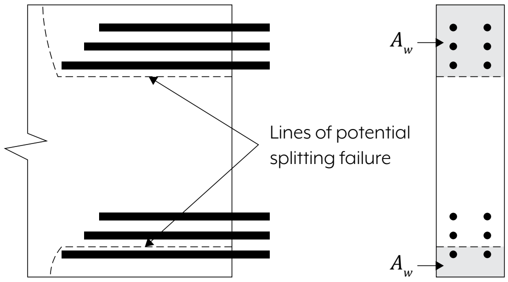


Figure 9. Cross Sectional Area of Portal Frame Member used in Calculating Wood Fracture Capacity

Rod Group Second Moment of Inertia

Determine the second moment of inertia I provided by the steel rods. This can then be used to find the tension force in the most extreme rod in order to check for steel yielding and pull-out of the individual rods.

Tension Force in Extreme Steel Rod

Calculate the tension force in the rod furthest from the members centroid.

$$N_t^* = \frac{M^* y_{max}}{I_{max}}$$

where

y_{max} = distance from centroid and cross sectional area of furthest rod in the group respectively

Yielding of Extreme Steel Rod

Using the equations above, check yielding of the furthest rod.

When checking yielding of threaded rods in tension, it is important to calculate the capacity of the rods based on their tensile area, which is smaller than the gross area of the rod.

Check furthest Rod for Pull Out Failure

$$Q^* \leq (\phi Q_n)_{pullout}$$

$$(\phi Q_n)_{pullout} = \phi_{conn} k_1 n k_g Q_k$$

where

ϕ_{conn} = 0.7 for wood fasteners other than nails or bolts (refer NZS 3603:1993, clause 2.5)

k_1 = load duration factor (refer NZS 3603:1993, table 2.4)

n = number of steel rods

k_g = rod group reduction factor (Table 16)

and

$$Q_k = 6.73 k_b k_e k_m \left(\frac{l}{d}\right)^{0.86} \left(\frac{d}{20}\right)^{1.62} \left(\frac{h}{d}\right)^{0.5} \left(\frac{e}{d}\right)^{0.5}$$

where

k_b = rod type factor

1.0 for threaded rods

0.8 for deformed rods

k_e = epoxy factor (taken as 1.0 for this guide)

1.0 for West System

1.0 for Araldite K-80

1.2 for Araldite 2005

k_m = moisture content factor

1.0 for moisture content < 15%

0.8 for moisture content 15 - 22%

l = embedment length ($5d \le l \le 20d$)

d = steel rod diameter ($12 \le d \le 24$ mm)

h = hole diameter ($1.15d \le h \le 1.4d$)

e = edge distance from centre of rod ($e \ge 2.5d$ recommended)

Table 16. Rod Group Reduction Factor, k_g

Number of Bars in Group	Reduction Factor k_g
1, 2	1.0
3, 4	0.9
5, 6	0.8

Check Shear in Steel Rods

At the interface between the timber members, shear forces must be transferred across the joint through the steel rods. The following condition from the steel standard, NZS 3404 must be satisfied:

$$V^* \le \phi V_f$$

$$\phi V_f = 0.62 \phi f_{uf} A_s$$

where

V^* = design shear force to be transferred across joint

ϕ = 0.8 for steel bolts or pins in shear

f_{uf} = minimum tensile strength for steel rod used in connection

A_s = cross sectional area of steel in joint (core area for threaded rods); effective areas of bolts and rods are given in Table 17.

Check Shear in the Joint

Due to the rapid build up of bending moments in the steel rods, significant shear forces can develop in the joint. There is little experimental data on shear in epoxy grouted rod type joints. The engineer shall use judgement when considering shear across the connection with consideration of the detailing in Figure10. The joint moment is assumed to develop over the spacing between steel rods and therefore the joint shear force is given by:

$$V^* = \frac{M^*}{s}$$

where

M^* = design moment in joint

s = spacing between rod group centroids

The shear capacity of the joint is given by the standard equation for the member's shear capacity from NZS 3603:1993.

$$\phi V_n = \phi k_1 k_4 f_s A_s$$

where

k_1 = load duration factor

k_4 = parallel support factor

f_s = characteristic member strength in shear

A_s = shear area, $2bd/3$

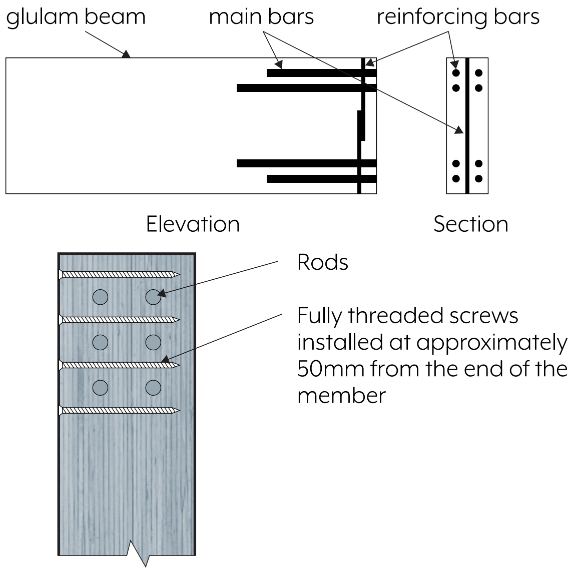


Figure 10. Top diagram is an example of reinforcing bars in Glulam. Bottom diagram is reinforcing LVL with threaded screws

To prevent splitting of portal members perpendicular to the grain, the use of transverse reinforcing is recommended. This consists of fully threaded screws crossing the potential split lines typically at 50 mm from the end of the member. There are no specific design criteria for transverse reinforcement. In these joints, however it is recommended that the cross sectional area of the transverse reinforcement be at least 1/25 of the area of the main rods (Buchanan and Moss, 1999).

It is also recommended that the rods have staggered embedment lengths of 75mm to reduce the potential splitting along the line of the rod and to reduce the stress concentrations at the end of the rods. It is recommended that the outer rods have the shorter length.

4.5.3 Long Term Behaviour

A study carried out by Fragiacomo et al. (2010) Into the long term behaviour of portal knee joints using epoxy grouted steel rods identified crushing of the portal frame members perpendicular to grain and deformations due to joint rotation to be problem areas. Joint rotation was found to contribute up to 50% of the frames' total deflection, a significant result when moment resisting joints are often assumed to be completely rigid and ignore the effects of joint rotation. The method of analysis outlined above is mechanics based and does not account for creep or long-term stress concentrations in the joint (Fragiacomo et al., 2010). Further research is required in order to develop a general design method for evaluating the stiffness of epoxied connections (Fragiacomo et al., 2010).

While research is ongoing, buildings which have utilised these joints in practice have performed satisfactorily. Glued in rods are most certainly a viable form of moment resisting joint, in light of recent research however it is prudent for designer to make allowances for long term reductions in the strength of the timber and deformations due to joint rotation.

With the use of the steel knee, all rods are installed parallel to the grain and therefore the crushing of the member perpendicular to the grain is avoided. By detailing a nut either side of the steel gusset flange all tension and compression loads are taken by the epoxy dowels.

It is not recommended to use epoxy grouted steel rods in structures subjected to high amplitude cycling moisture content (≈ 10% variation). There is a tendency for micro-cracks to develop in the wood with time which can increase in size and result in longitudinal splits.

4.5.4 Table of capacities

Table 17. Glulam GL8 Moment Connection Capacities - Epoxy Rods (MKCER)

Member Size D x B (mm)	Member capacity $f_b = 19MPa$ $ØMn$ (kNm)	Rod diameter (mm)	No of rods at the end of each member	No of Rows	Top distance and row spacing (mm)	Edge distance (mm)	Minimum embed-ment depth (mm)	Joint Capacity $ØMn$ $k_1 = 1$ (kNm)
450 x 90	46	12	3	3	45	45	250	29
450 x 135	69	12	6	3	40	40	250	40
540 x 90	66	12	3	3	45	45	250	37
540 x 135	100	12	6	3	45	40	250	54
630 x 90	90	12	3	3	50	45	250	49
630 x 135	136	12	6	3	50	40	250	74
630 x 180	181	12	6	3	50	45	250	100
720 x 90	118	12	3	3	60	45	250	67
720 x 135	177	12	6	3	60	40	250	102
720 x 180	236	12	6	3	60	40	250	138
810 x 90	150	12	4	4	65	45	250	103
810 x 135	224	12	8	4	65	40	250	156
810 x 180	299	12	8	4	65	45	250	210
900 x 90	185	12	4	4	65	45	250	117
900 x 135	277	12	8	4	65	40	250	178
900 x 180	369	12	8	4	65	40	250	242

Table 18. LVL 13 Moment Connection Capacities - Epoxy Rods (MKCER)

Member Size D x B (mm)	Member capacity $f_b = 45MPa$ $ØM_n$ (kNm)	Rod diameter mm	No of rods each end of member	No of Rows	Top distance and row spacing	Edge distance (mm)	Minimum embed-ment depth mm	Joint Capacity $ØM_n$ (kNm)
450 x 63	66	12	3	3	40	31.5	250	44
450 x 90	95	16	3	3	45	45	350	72
450 x 135	142	16	4	2	45	40	350	83
600 x 90	161	16	3	3	45	45	350	104
600 x 135	241	16	6	3	45	40	350	156
600 x 180	322	16	6	3	45	50	350	207
800 x 90	272	16	3	3	45	45	350	148
800 x 135	409	16	6	3	45	40	350	220
800 x 180	545	16	6	3	45	50	350	292
900 x 90	338	20	3	3	55	45	400	198
900 x 135	507	20	6	3	55	40	400	295
900 x 180	676	20	6	3	55	50	400	380
1200 x 90	573	20	4	4	55	45	400	346
1200 x 135	859	20	8	4	55	40	400	511
1200 x 180	1146	20	8	4	55	45	400	655

4.6 Moment Knee Connections – Quick Connect (MKCQC)

The "Quick-Connect" moment connection was developed at the University of Auckland with the objective to minimise onsite work. This system is a moment resisting joint utilising rods placed at the member extremities. The bending moment is transferred across the joint by a force couple carried by steel rods housed in timber sleeves. (Figures 12-18).

The timber sleeves and bearing block can be pre-fabricated leaving only the rods and steel end bearing plates to be fixed on site. The onsite time is reduced with potential saving on labour and cranage time.

The timber sleeves are fixed with fully threaded screws at 60° to the load. This angle results in a reduced demand on the screws and a stiffer connection overall. The steel bearing plate at the end of the timber sleeve transfers the force from the steel rod to the sleeve. Shear is transferred through separate dowels fitted between the members.

The Quick-Connect joint may also be used for connections to steel or to concrete.

Quick Connect Knee Portal

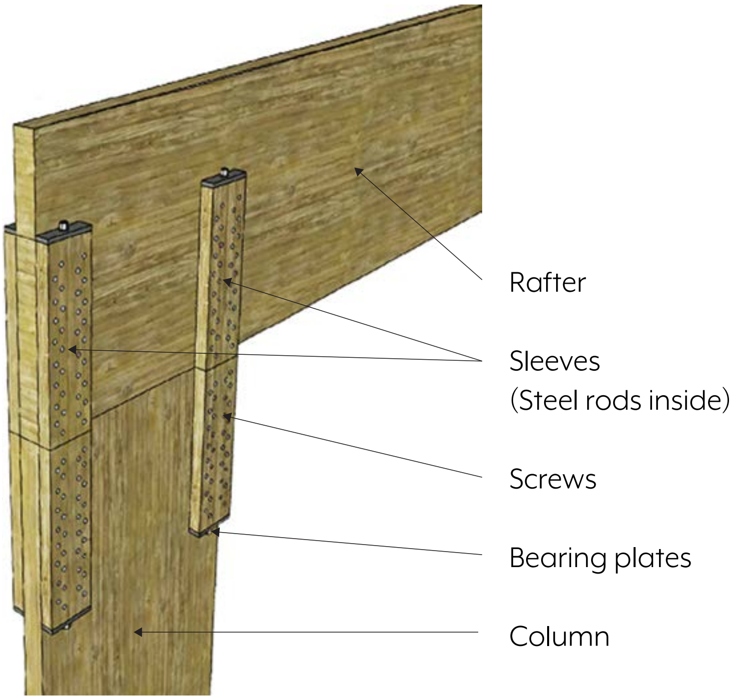


Figure 11. Quick-Connect Moment Connection Knee Joint Application

Figure 12. Quick-Connect Column to Foundation Connection

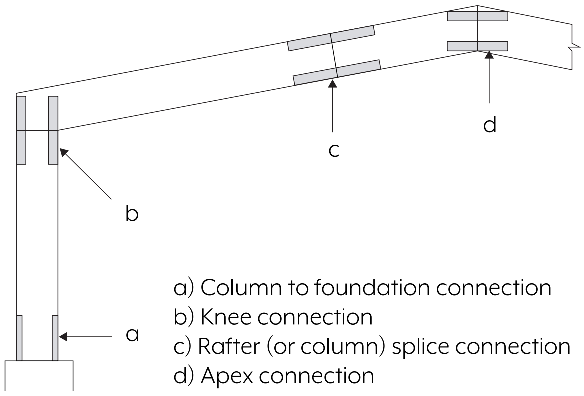


Figure 13. Applications of the Quick-Connect Moment Connection in Portal Frame Construction

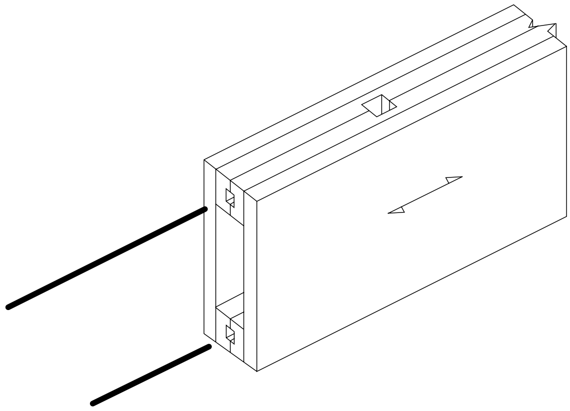


Figure 14. Quick-Connect Moment Connection Utilised in a Box Beam

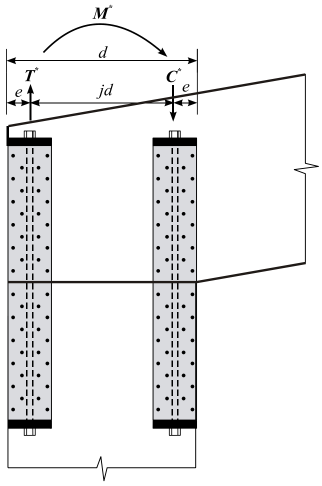


Figure 15. Quick-Connect Moment Connection Mechanics

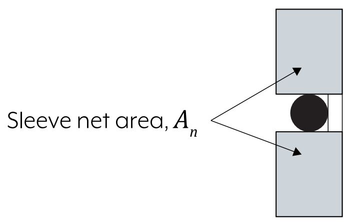


Figure 16. Net Area of Timber Sleeve Acting in Compression and Bearing

4.6.1 Design Procedure

The design method presented is for a portal knee connection. The process can easily be adapted for other connections at the apex, column base and beam splice. The connections action works by assuming a force couple between the rods in tension and bearing on at the front of the member.

4.6.2 Design Portal Frame Members assuming the Joint is rigid, $k_{rot} = \infty$

It is assumed in this case that a preliminary member design has been carried out and approximate member sizes established.

4.6.3 Design of Connection Detail

The sleeves are sized to allow for screw spacing and end distances.

The sleeves are designed to resist the compression force applied to their end grain. Calculate the moment arm jd of the rods as follows:

$$jd = d - 2e$$

where

jd = moment arm between the steel rods, as shown in Figure 15.

d = width of the main member

e = distance from the extreme fibre to the centre of the rod, as shown in Figure 15

Rod Equation

The equation below allows preliminary calculations for tension and compression forces in each set of rods. Only one set of the rods will act in tension with compression taken by the timber. This loading is reversed when the applied actions are reversed. The sleeves are sized for axial compression loads resulting from the applied moment as well as the tension force in the member. The force per rod is then calculated from:

$$C^*_{sleeve} = T^*_{rod} = \frac{\frac{M^*}{k_1}}{2jd} + \frac{\frac{N^*}{k_1}}{4}$$

Design of Sleeves

The timber sleeve must be capable of resisting a compression force equal to the tension force in the rod. The compression capacity is checked in accordance with NZS 3603:1993 assuming the sleeve is fully restrained therefore k_8 = 1.

$$C^*_{sleeve} \leq \phi N_{nc,sleeve}$$

$$\phi N_{nc,sleeve} = \phi k_1 f_c A_n$$

where

ϕ = strength reduction factor for sleeve

= 0.9 for LVL or 0.8 for timber and glulam

k_1 = load duration factor

f_c = compression strength, refer to for the characteristic stresses

A_n = net cross sectional area of sleeve acting in compression. Refer to Figure 16.

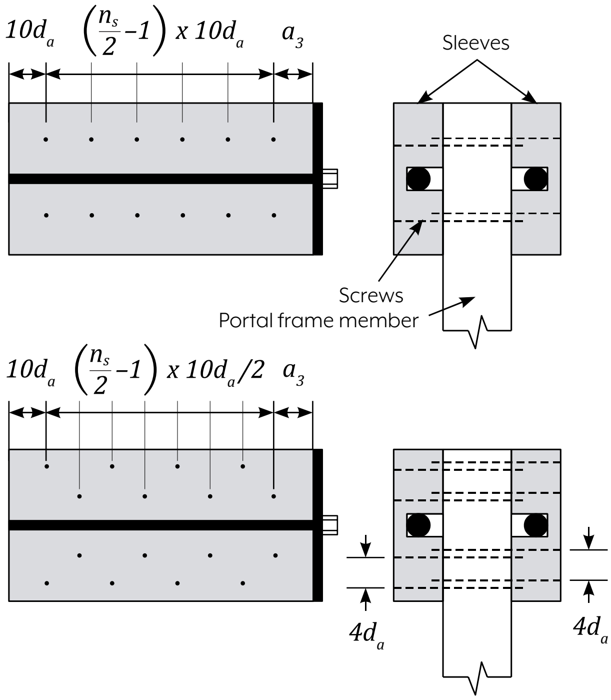
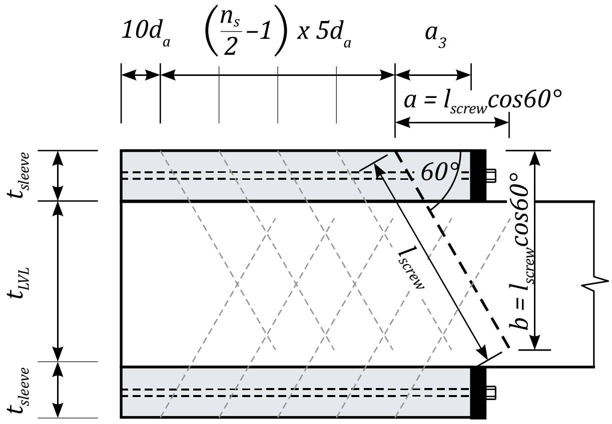


Figure 17. Screw Spacing Requirements for Quick-Connect Moment Connection, side view



$$a_3 = \begin{cases} a + 50mm & for\ b > (t_{sleeve} + t_{LVL}) \\ 10d & for\ b < (t_{sleeve} + t_{LVL}) \end{cases}$$

Figure 18. Screw End Distance Requirements for Quick-Connect Moment Connection, top view

The sleeve size will also depend on the arrangement of the screws and whether there is a single or double row of screws each side of the rod. Edge distances of 5d will govern the sleeve size and this in turn will affect the lever arm and overall moment capacity of the joint.

Design of Screws

Various screw capacities P_{screw} have been tested and are given in Tables 19 & 20. The capacity is different depending on orientation to the grain.

The following equation is used to determine the number of screws required to resist the tension and compression forces at each timber sleeve:

$$\frac{T_{rod}}{Øk_1 P_{screw}} = n_s$$

where

n_s = number of screws per sleeve required to resist tension forces in the connection

T^*_{rod} = tension force carried by each steel rod

P_{screw} = 5th percentile strength of a single screw as given in Table 19 & 20

Rows of screws along the length of the sleeve should be staggered with a minimum spacing of 3d between rows. To ensure screws do not interfere with those on the opposite side of the connection, spacings between rows of 3d and 5d are used on alternate sides. Edge distance of 5d will likely govern the width of the sleeve.

The screw characteristic load capacity depends on grain orientation as shown in Figure 21. Different numbers of screws may be required for the column and rafter depending on the screw type used.

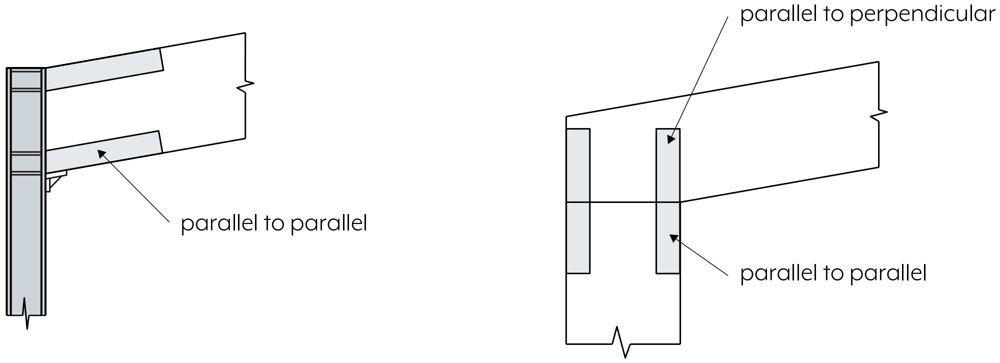


Figure 19. Grain Orientations for Screw Design

TABLE 19. GLULAM Screw Types and 5th Percentile Capacities taken from Expan Portal Design Guide

Screw Type	Sleeve thickness (mm)	GLULAM	
		Parallel (kN)	Perp (kN)
Würth Assy plus chse hd scr Aw30 Ø6/L200	45 90	4.6 8.1	5.1 9.2
Würth Assy plus vmp hd scr Aw40 Ø8/L140	45 90	4.0 7.0	4.4 8.7
Würth Assy plus vmp hd scr Aw40 Ø8/L200	45 90	4.1 11.6	8.1 12.2
Würth Assy plus vmp hd scr Aw40 Ø10/L200	45 90	8.6 13.7	12.2 13.6
Würth AMO III typ 2 Aw30 Ø7.5/L200	45 90	4.9 8.2	3.6 7.2
Spax timber screw Ø8/L200	45 90	9.4 11.8	6.9 11.4
Spax timber screw Ø10/L200	45 90	12.5 15.3	9.5 12.1

TABLE 20. LVL Screw Types and 5th Percentile Capacities taken from Expan Portal Design Guide

Screw Type	Sleeve thickness (mm)	LVL	
		Parallel (kN)	Perp (kN)
Würth Assy plus chse hd scr Aw30 Ø6/L200	45 63	8.5 9.8	8.2 10.1
Würth Assy plus vmp hd scr Aw40 Ø8/L140	45 63	9.7 13.1	11.4 10.9
Würth Assy plus vmp hd scr Aw40 Ø8/L200	45 63	10.6 14.7	11.6 13.6
Würth Assy plus vmp hd scr Aw40 Ø10/L200	45 63	13.8 18.2	13.7 19.8
Würth AMO III typ 2 Aw30 Ø7.5/L200	45 63	6.7 9.2	6.9 9.7
Spax timber screw Ø8/L200	45 63	9.5 14.4	11.7 13.8
Spax timber screw Ø10/L200	45 63	12.9 16.5	12.4 17.1

Check Block Tear-out Resistance in Main Portal Member

The sleeves must be large enough to prevent block tear-out type failure of the main member. It is assumed the two properties are dependant as tearing will only occur if shear has occurred and vice versa.

First calculate the shear resistance as:

$$\phi V = \phi k_1 f_s A_s$$

where

k_1 = Load duration factor

f_s = shear resistance of portal frame members. Refer to for characteristic stresses of glulam and LVL.

A_s = shear area

The shear area A_s is:

$$A_s = t \cdot (l_s - a_3)$$

where

t = thickness of the main member being considered

l_s = length of the timber sleeve

a_3 = end distance, see Figures 17 & 18

The dimension ls is taken as the length of the sleeve associated with the parallel-to-grain member. The sleeve length sized for the number of screws at the recommended spacings.

The tearing resistance of the main member is given by:

$$\phi T = \phi k_1 f_t A_t$$

where

f_t = tensile strength of main portal frame member.
Refer to for the characteristic stresses of glulam and LVL

A_t = tensile area

and

$$A_t = t \times (w_{sleeve} - edge\ distance)$$

where

w_{sleeve} = depth of timber sleeve

t = thickness of main member

The resistances obtained from equations above are then added to give the final tearing and shear resistance of the main member where the timber sleeves are attached.

$$2T^*_{rod} \leq \phi T + \phi V$$

Bearing Plate Design

The bearing plates are designed assuming the timber sleeve consists of two individual pieces. Figure 18 shows the two hatched areas that are considered for bearing, the white area is neglected. This approach is conservative.

To specify a bearing plate length first calculate the required bearing area for ultimate limit state design criteria using:

$$A_p \geq \phi \frac{T^*_{rod}}{k_1 f_c}$$

where

T^*_{rod} = tension force carried by each steel rod

k_1 = load duration factor

f_c = characteristic compression strength of timber of sleeves

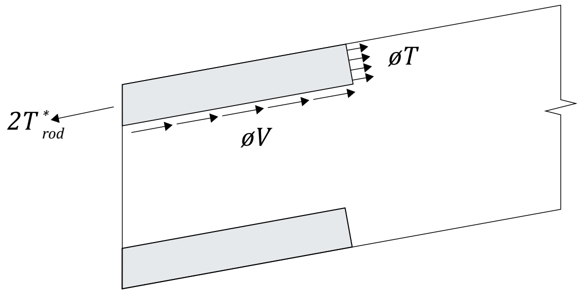


Figure 20. Combined Shear and Tearing in Portal Frame Rafter

The plate size is limited by the timber sleeve dimension with plate thickness calculated considering both serviceability and ultimate limit cases. Plate deflection is likely to govern the design and a deflection limit of 0.1 mm is recommended. Small deformations at the knee connections can lead to large displacement in the overall structure however the plate deflection limit is at the discretion of the designer.

Plate deflection is calculated assuming a beam with two cantilevers mirrored at the rod axis.

$$\Delta = \frac{wl^4}{8EI}$$

where

w = uniformly distributed load acting on bearing plate = $\frac{T^*_{rod}}{2l}$

l = outstanding length of bearing plate either side of the rod see Figures 21 & 22

E = modulus of elasticity of steel bearing plate

and the moment of inertia I is:

$$I = \frac{bt^3}{12}$$

where

b = height of the bearing plate (mm), see Figure 22.

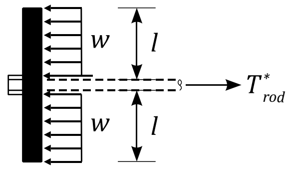


Figure 21. Bearing Plate Double Cantilever Action on Bending

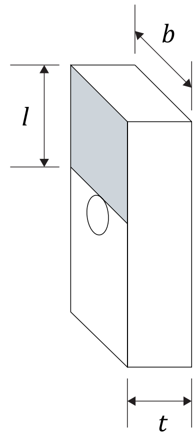


Figure 22. Bearing Plate Dimensions

t = thickness of bearing plate (mm)

Check the plate satisfies strength requirements using:

$$t_u \geq \sqrt{\frac{3lT^*_{rod}}{2\phi f_y b}}$$

Typically the deflection limit of 0.1mm recommended will govern the design and therefore grade 300 steel plate is typically sufficient

Rod Design

The rods are initially sized for tension strength only. Rods can be either grade 4.6 or 8.8. Grade 8.8 rods are more efficient for strength.

The tension resistance of the rod must be larger than or equal to the applied tension force:

$$T^*_{rod} \leq \phi N_{tf}$$

where N_{tf} is the tension resistance of the rod given by:

$$N_{tf} = A_s f_{uf}$$

Where

ϕ = 0.8

A_s = tensile area of the rod.

f_{uf} is the ultimate tensile strength: 860 MPa for grade 8.8 and 400 MPa for grade 4.6 rods

Table 21. G8.8 Threaded Rod Capacity

Rod diameter (mm)	A_s (Tensile stress area mm^2)	Factored rod capacity in tension (kN)
12	84.3	58.0
16	157	108.0
20	245	168.6
24	353	242.9
30	561	386.9

Nuts should be tightened to finger tight plus a quarter turn. Further tightening will result in a stiffer connection, but excessive tensioning will reduce the residual load capacity.

Experience has shown that there may be some relaxation in the joint after erection and it is recommended that the rods are checked and retightened.

4.6.4. Evaluation of rotational stiffness k_{rot}

The term k_{rot} is the rotational stiffness of the moment connection. It is assumed that deformations will remain in the elastic range. The moment is calculated as part of the preliminary design for the portal frame structure. The rotational stiffness of the joint is:

$$k_{rot} = \frac{M^*}{\theta}$$

M* is the design bending moment in the joint. The rotation θ is given by the following equation:

$$\theta = \frac{\Delta_{rodass}}{jd}$$

Where the deflection of the rod assembly Δ_{rodass} may be calculated by:

$$\Delta_{rodass} = \Delta_1 + \Delta_2 + \Delta_3 + \Delta_4$$

where

$$\Delta_1 = \frac{T^*_{rod} L_{rod}}{E_{rod} A_{rod}}$$

$$\Delta_2 = \frac{k_{37} T^*_{rod} L_{s,total}}{2E_{sleeve} A_{sleeve}}$$

$$\Delta_3 = \frac{k_{37} T^*_{rod}}{k_{s,perp} n_{s,perp} + k_{s,para} n_{s,para}}$$

$k_{s,perp}$ = 7550N/mm, $k_{s,para}$ = 9650N/mm (Stiffness of screw connection per unit length)

Δ_4 = set bearing plate deformation

Equations above are both based on the elastic deformation of a member under axial load. This is derived from the standard deflection formula:

$$\Delta = \frac{PL}{EA}$$

For the deformation of a single rod or timber sleeve, the coefficient P may be substituted for the tension force calculated using the rod equation T^*_{rod}

$$P = T^*_{rod}$$

The deflection due to crushing of the LVL sleeve is a function of the total sleeve length over both the rafter and the column. The compression load acting in the LVL sleeve decreases from a maximum value (taken as T^*_{rod}) at the first screw nearest to the steel bearing block to zero at the interface between the rafter and column.

Return to step I. and determine if the bending moment obtained using the given level of rotation is different to the moment obtained from the original assumption $k_{rot} = \infty$

Having completed the initial estimates above determine if there is a difference between the moment calculated assuming that $k_{rot} = \infty$ and the moment obtained using the k_{rot} value. This is done with the help of a structural analysis program.

Adjust Member Sizes and Connection Design accordingly

If an abnormality has been found in the calculation then the designer should use an iterative approach (with the help of a spreadsheet) to determine a connection size which gives comparative results.

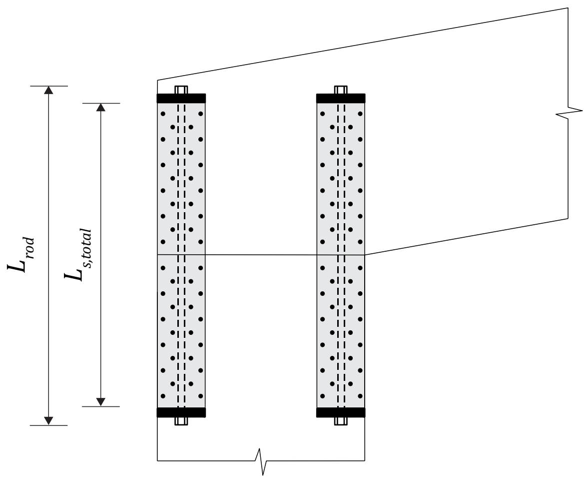


Figure 23. Dimensions Used in Determining the Deflection of the Rod Assembly

4.6.5. Table of Capacities

Capacities have been calculated using a spreadsheet developed by the University of Auckland.

Grade 8.8 rod is used for design. The bearing plate is grade 300MPa for 0.1mm deflection limit. A k_l=1 has been assumed for these connection capacities. The thickness of the bearing plate given is the minimum and actual thickness will depend on availability of plate.

Table 22. Glulam GL8 Moment Connection Capacities Quick Connect (MKCQC)

Member size	Member capacity ØMn f_b = 19MPa (kNm)	Rod size G 8.8	Bearing plate minimum thickness (mm)	Sleeve size GL8 (mm)	Screw type	Number of screws Column	Number of screws Rafter	Joint capacity ØMn (kNm)
450x90	46	12	22	140x45	6Ø x 140mm	14	12	30
540x90	65	16	23	140x45	6Ø x 140mm	18	16	50
630x90	90	16	24	140x45	6Ø x 140mm	24	22	85
720x90	118	16	26	140x45	6Ø x 140mm	26	22	105
810x90	149	16	26	140x45	6Ø x 140mm	26	22	120
810x135	224	20	33	190x90	8Ø x 200mm	16	16	175
810x180	299	24	37	190x90	8Ø x 200mm	24	24	270

900x90	184	20	25	140x45	6Ø x 140mm	26	22	135
900x135	227	20	32	190x90	8Ø x 200mm	16	16	200
900x180	269	24	35	190x90	8Ø x 200mm	22	22	260

Table 23. LVL 13 Moment Connection Capacities - Quick Connect (MKCQC)

Member size	Member capacity ØMn f_b = 45MPa (kNm)	Rod size Gr 8.8	Bearing plate minimum thickness (mm)	Sleeve size LVL13 (mm)	Screw type	Number of screws Column	Number of screws Rafter	Joint capacity ØMn (kNm)
450x90	95	16	26	140x45	6Ø x 140mm	12	12	45
600x90	161	16	30	140x45	6Ø x 140mm	16	16	90
800x90	272	20	40	200x63	8Ø x 140/200mm	16	16	190
800x135	409	24	45	200x63	8Ø x 200mm	20	22	270
800x180	545	30	45	200x63	8Ø x 200mm	24	26	320
900x90	338	20	40	200x63	8Ø x 200mm	14	16	220
900x135	507	24	45	200x63	8Ø x 200mm	20	20	300
900x180	676	30	47	200x63	8Ø x 200mm	28	28	420
1200x90	573	24	53	240x75	10Ø x 200mm	32	32	400
1200x135	859	30	62	240x75	10Ø x 200mm	28	28	700
1200x180	1146	36	64	240x75	10Ø x 200mm	34	32	835

4.7 Moment Knee Connections – WS Dowel and Steel Plate (MKCWSS)

This joint is often used where appearance is important. In New Zealand there have been several buildings constructed using this system.

WS dowels come in 7mm diameter and are self-drilling into the timber and through steel plates so pre-drilled holes are not needed. The dowels can be drilled through a G250 steel plate, up to 3 steel plates at 5mm thick or a single plate up to 10mm thick. The dowels are finished slightly recessed into the timber surface.

The dowels require some experience for installation. Specialist drilling equipment is needed (which is available from the supplier) to ensure the dowels are installed perpendicular to the member. Once mastered the dowels can be relatively efficiently installed. The portals should be laid flat to install the dowels vertically. It is difficult to successfully drill the dowels with the portal standing.

Drilling with handheld tools can be done although there is a higher risk of the dowels breaking during the process.

As the dowels are designed to drill through grade 235 steel plate the use of grade 250 steel as available in New Zealand is recommended. Grade 300/350 plate steel is too hard for the dowels and the drilling is difficult with a high risk of tip breakage.

Because the dowels are self-drilling, they create a stiff connection and have inherent fire resistance as the steel (all but the heads of the dowels) are typically protected by timber.

The load capacity of the dowels has been developed using the European Yield Model and verified by testing in Switzerland where the dowels are manufactured.

WS Dowel knee connection

Table 24. Dowel Lengths

Timber thickness mm	Dowel length mm
90	73
135	133
180	173

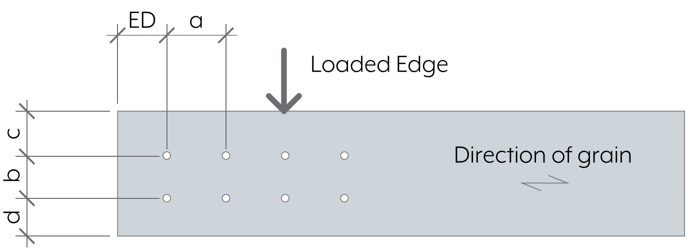


Figure 24. Dowel Spacing

Table 25. Minimum Dowel Spacing

		Minimum distance (mm)
End distance	ED	80
Spacing along the grain	a	50
Spacing across the grain	b	20
Loaded edge	c	30
Unloaded edge	d	20

4.7.1. Geometric Recommendations

Allow for 8mm (or greater) slot in the timber for the steel plate. This is a standard minimum kerf thickness for a chainsaw cut when shaped using CNC machine.

Use the published table of capacities for the dowels depending on number of plates and timber thickness and angle of load to the grain. Dowel lengths will need to be less than the overall timber thickness.

The steel plate should be at least 10mm smaller than the timber to avoid the plate projecting past the timber edge. A minimum distance of 20mm from the dowel centreline to the edge of the steel plate is recommended.

A spacing of 30mm across the grain has been adopted for this guide. For LVL and larger glulam sizes it is suggested that the dowel spacing be increased to 70mm along the grain. This reduces the total number of dowels required as the lever arm between the end groups is increased. It also reduces the potential for the timber splitting along the line of dowels.

4.7.2. Design Actions

As for the other types of connections, the design actions to be resisted by the joint are obtained from the structural analysis of the portal frame. Rigid connections can be assumed. Bending will dominate the connection design, however the effect of the axial and shear forces in the joint must still be considered.

For knee joints, it is important to ensure that the appropriate axial and shear forces are used in the design. The critical section for the plate design is at the line where the column centreline intersects the rafter soffit. While the design bending moment is the same for both the rafter and column sections, the axial and shear forces should be taken from the column design actions.

To check the strength of the plate the design bending moment, axial and shear force should be taken at the point of intersection between the column centreline and the rafter soffit. Design actions for the dowels should be taken at the centroid of the dowel group.

4.7.3. Design Procedure

The characteristic shear capacity of the dowels (R_k) is determined from Table 26. The capacity varies depending on the length of the dowel, the angle of the applied force to the grain and the number of shear planes and is based on the European terminology.

For this guide the adopted conversion factor for timber strength class has been taken as = 1.0 for both Glulam and LVL. For LVL this is conservative. For LVL with a characteristic density of 480 kg/m^3, the factor of 1.1 may be used from Table 26.

The equation for dowel capacity:

$$\text{Ø}Q_n = \text{Ø}k_1 n_{ef} R_k$$

Where

Ø = 0.8 (maybe should be 0.75) (1/1.3)

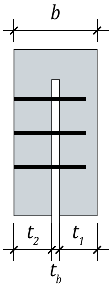
k_1 = load duration factor as per NZS3603

n_{ef} = effective number of fasteners

R_k = Characteristic shear capacity of the dowel for the appropriate length, number of shear planes and angle to the grain.

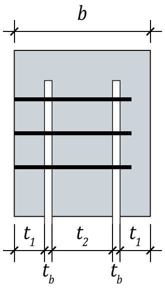
In the case of several dowels n inserted in rows in the direction of the grain, the characteristic values per dowel R_k must be multiplied by the effective number n_{ef} in accordance with Table 27. Effective number n_{ef} is applicable for a spacing of 50mm between the dowels in the direction of the grain.

Table 26. WS Dowel Characteristic Capacities (Taken from www.rothoblaas.com)



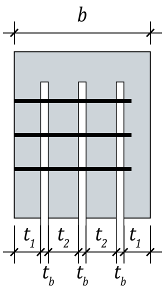
Characteristic load-carrying capacity R_k in kN **per dowel** in **double shear** conections Table 2

fastner	WS-T	7 x 73	7 x 93	7 x 113	7 x 133	7 x 153	7 x 173	7 x 193	7 x 213	7 x 233
timber width	b in mm	80	100	120	140	160	180	200	220	240
side member	t_1 in mm	34	44	54	64	74	84	94	104	114
middle member	t_2 in mm	–	–	–	–	–	–	–	–	–
angle **α** between the force and the direction of grain	0°	7.15	8.41	9.51	10.6	11.3	11.7	11.7	11.7	11.7
	30°	6.54	7.80	8.74	9.88	10.5	11.1	11.1	11.1	11.1
	45°	6.04	7.29	8.10	9.11	9.84	10.4	10.6	10.6	10.6
	60°	5.62	6.87	7.58	8.47	9.29	9.80	10.1	10.1	10.1
	90°	5.27	5.52	7.13	7.93	8.82	9.28	9.71	9.71	9.71



Characteristic load-carrying capacity R_k in kN **per dowel** in **fourfold shear** conections Table 3

fastner	WS-T	7 x 73	7 x 93	7 x 113	7 x 133	7 x 153	7 x 173	7 x 193	7 x 213	7 x 233
timber width	b in mm	80	100	120	140	160	180	200	220	240
side member	t_1 in mm	–	–	–	40	40	55	65	65	75
middle member	t_2 in mm	–	–	–	48	68	58	58	78	78
angle **α** between the force and the direction of grain	0°	–	–	–	17.8	19.8	21.3	22.4	22.4	23.1
	30°	–	–	–	16.3	18.6	19.4	20.5	21.1	21.6
	45°	–	–	–	15.0	17.6	17.8	18.8	19.8	20.5
	60°	–	–	–	13.9	16.8	16.4	17.3	18.7	19.5
	90°	–	–	–	13.0	15.8	15.3	16.1	17.7	18.6



Characteristic load-carrying capacity R_k in kN **per dowel** in **sixfold shear** conections Table 4

fastner	WS-T	7 x 73	7 x 93	7 x 113	7 x 133	7 x 153	7 x 173	7 x 193	7 x 213	7 x 233
timber width	b in mm	80	100	120	140	160	180	200	220	240
side member	t_1 in mm	–	–	–	–	–	39	39	43	53
middle member	t_2 in mm	–	–	–	–	–	42	52	58	58
angle **α** between the force and the direction of grain	0°	–	–	–	–	–	25.0	29.1	31.7	32.8
	30°	–	–	–	–	–	22.8	26.4	28.8	29.8
	45°	–	–	–	–	–	20.9	24.2	26.4	27.2
	60°	–	–	–	–	–	19.3	22.3	24.4	25.0
	90°	–	–	–	–	–	17.8	20.6	22.6	23.2

ρ_k in kg/m^3	350	380	410	430	450
conversion factor	0.93	1.00	1.04	1.06	1.09

The calculation of the design value R4 is performed in accordance with EN 1995-1-1:2004/A1. Section 2:

$$R_d = \frac{R_k \bullet k_{mod}}{\gamma_M}$$

with γ_M = 1.3 in accordance with EN 1995-1-1:2004/A1. Table 2.3

Figure 25 below illustrates a basic joint arrangement. The theoretical moment capacity can be determined by considering each dowel individually each acting at a different angle to the grain and radius about the centre of the group. This method could be somewhat time consuming and two simpler approaches are presented below.

Method 1. Equivalent circle.

Dowels can be analysed assuming an equivalent circle similar to the nailed gusset design. The average radius of the dowel group can be calculated manually or determined graphically. The dowel capacity $ØQn$ is calculated assuming all dowels act at 45° to the grain. The joint moment capacity is then:

$$ØM_n = ØQ_n \times no\ of\ dowels \times R_{avg}$$

Method 2. Moment couples.

A further approach is to use 2 dowel groups that act as couples illustrated the figure below. The end group is acting perpendicular to the grain and the side group acting parallel.

For dowels acting perpendicular (90°) to the grain the factor n_{ef} = 1.0 can be used. For the dowels acting parallel to the grain the factor for 5 or more dowels n_{ef} is taken as 3.66/5.00 = 0.732.

The joint moment capacity is then the sum of the couples.

Method 2 if the approach taken for this guide. LVL members may require additional capacity with two rows of dowels at the ends as per Figure 27.

The two methods above gives similar results and are considered accurate enough for most design purposes.

Table 27. Effective Number of Fastners n_{ef}

Effective number of fastners n_{ef}

	n	1	2	3	4	5
angle α between the force and the direction of grain	0°	1.00	1.61	2.31	3.00	3.66
	30°	1.00	1.74	2.54	3.33	4.11
	45°	1.00	1.80	2.66	3.50	4.33
	60°	1.00	1.87	2.77	3.67	4.55
	90°	1.00	2.00	3.00	4.00	5.00

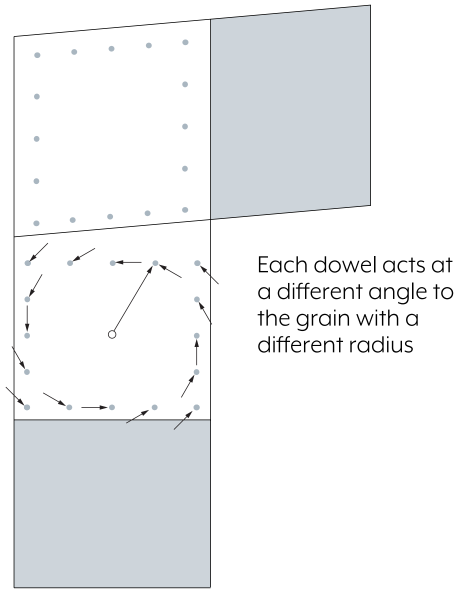


Figure 25. Individual Dowel Analysis in Basic Arrangment

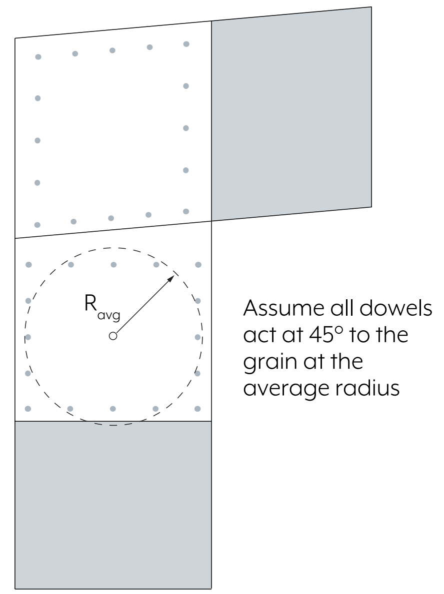


Figure 26. Equivalent Circle

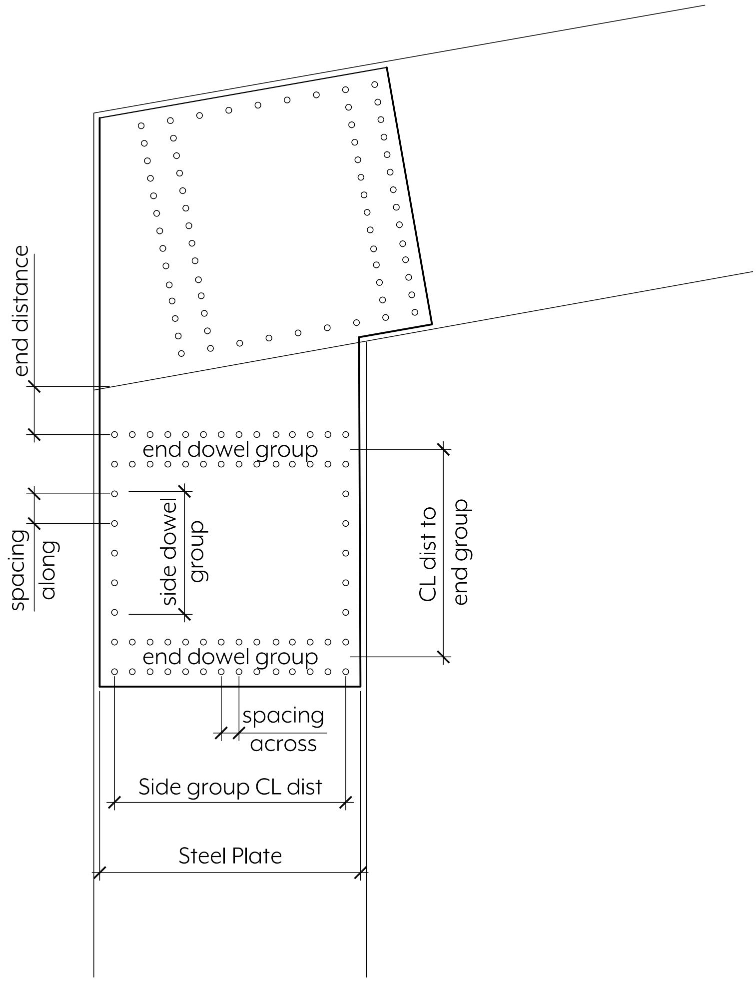


Figure 27. WS Dowel Moment Couples

STEEL PLATE CAPACITY

The capacity of the steel plate is determined from steel design principles. For the moment capacity the gross section is used. For tension and shear allowance is made for the reduced section with the dowel holes removed from the area.

$ØM_n = Øf_y Z$

$Ø$ = 0.9 and f_y = 250 MPa.

The plate can be assumed to be fully restrained. Buckling is resisted by the action of the dowels and timber.

It is recommended that the dowel capacity is greater than the plate so any potential yielding occurs in the plate first. This also allows for extra capacity should any of the dowels break during driving.

The shear and axial load capacities can also be calculated using steel design principles.

Worked example

600 x 90 LVL 13: Rafter and column portal

Timber bending capacity

$ØM_u = Ø\, k_1\, k_8\, k_{24}\, Z.f_b$

Where $Ø$ = 0.9 for LVL

k_1 = 0.6 for permanent loads

0.8 for live load combinations

1.0 for wind and earthquake load combinations

k_8 = stability factor ~ taken as 1.0 for this case

k_{24} = size factor

$= \left(\frac{95}{D}\right)^{0.167} = \left(\frac{95}{600}\right)^{0.167} = 0.725$

$Z = \frac{b_{eff}\, d^2}{6}$

b_{eff} = effective width = 90mm – 8mm slot for plate

→ $Z = 0.82 \times 0.6^2 / 6 = 4.92 \times 10^{-3}\ m^3$

F_b = 45 MPa for LVL 13

→ $Ø\,M_u = 0.9 \times k_1 \times 0.725 \times 4.92 \times 10^{-3} \times 45 \times 1000$

= 144 kNm for $k_1 = 1.0$

Capacity for steel plate – plate size 590 x 6 m

Bending capacity plate

$ØM_u = Ø\, fy\, Z$

$Ø$ = 0.9 for steel

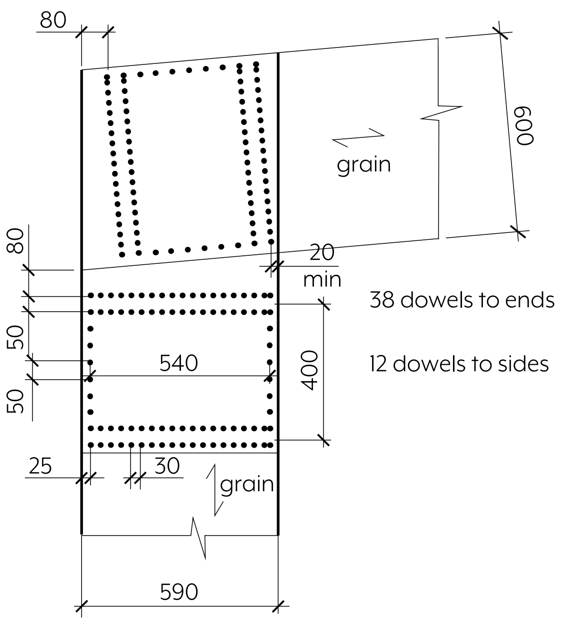
f_y = 250 MPa

$Z = bd^2 / 6$

$= 0.006 \times 0.59^2 / 6 = 3.48 \times 40^{-4}\ m^3$

$ØM_u$ plate = $0.9 \times 250 \times 3.48 \times 10^{-4} \times 1000$

= 78 kNm



JOINT PLATE ARRANGEMENT

For 90mm width timber use 73mm length dowels

From Table 27

Angle to grain 0° R_K = 7.15 kN $n_{eff} = \frac{3.66}{5}$ side group

90° R_K = 5.27 kN n_{eff} = 1.0 end group

For end dowels acting at 90° to grain

$$n_{eff} = \frac{5}{5} = 1.0$$

Centreline distance between end groups = 0.40 m (lever arm)

38 dowels each end

→ moment capacity for end dowel group

$$\varnothing M_u \, ends = \varnothing \, R_K \times n_{eff} \times \text{No of dowels} \times \text{lever arm} \times K_1$$

$$= 0.8 \times 5.27 \times 10 \times 38 \times 0.40$$

$$= 64 \text{ kNm}$$

Dowels activity 0° to grain – side group 6 dowels each side

$$n_{eff} = 3.66 / 5.0 = 0.732$$

Centre line distance between end groups = 0.54m

$$\varnothing M_u = 0.8 \times 7.15 \times 0.723 \times 6 \times 0.54$$

$$= 13.4 \text{ kNm}$$

→ total joint capacity = 64 + 13.4

= 77 kNm

4.7.3. Table of Capacities

The following tables summarise the joint moment capacities for a steel plate/WS dowel connection for Glulam and LVL. The load duration factor k_1 = 1 is used for the bending capacities given. The steel plate bending capacity is based on f_y = 250MPa The member capacity allows for the 8mm slot.

4.7.4 Table of Capacities

Table 28. Glulam GL 8 Moment Connection Capacity - WS Dowel and Steel Plate (MKCWSS)

Member Size	Member capacity f_b = 19MPa	Plate size	Dowel length	End dowels			Side Dowels		Total dowel no	Dowel Bending capacity	Steel plate bending capacity
(mm)	$ØM_n$ (kNm)	(mm)	(mm)	No of rows	No each end	Spacing (mm)	No each side	Spacing (mm)		$ØM_n$ (kNm)	$ØM_n$ (kNm)
450 x 90	47	430x5	73	1	14	30	8	50	44	40	35
540 x 90	68	520x5	73	1	17	30	9	50	52	54	51
630 x 90	93	610x5	73	1	20	30	10	50	62	77	70
720 x 90	121	700x5	73	1	23	30	11	70	66	102	92
810 x 90	153	790x5	73	1	26	30	11	70	74	127	117
900 x 90	189	880x5	73	1	22	40	14	70	74	157	145
900x135	275	2/880x5	133	1	22	40	12	70	68	313	290

Table 29. LVL 13 Moment Connection Capacity - WS Dowel and Steel Plate (MKCWSS)

Member Size	Member capacity f_b = 45MPa	Plate size	Dowel length	End dowels			Side Dowels		Total dowel no	Dowel Bending capacity	Steel plate bending capacity
(mm)	$ØM_n$ (kNm)	(mm)	(mm)	No of rows	No each end	Spacing (mm)	No each side	Spacing (mm)		$ØM_n$ (kNm)	$ØM_n$ (kNm)
450 x 90	86	440x 6	73	2	28	30	5	50	66	49	44
600 x 90	146	590 x 6	73	2	38	30	5	70	86	90	78
600x135	213	2/590x6	133	2	28	40	5	70	66	170	157
800x90	248	790 x 6	73	2	44	40	10	70	96	165	140
800x135	360	2/790x6	133	2	38	40	8	70	95	337	281
900x90	308	890 x 6	73	2	44	40	10	70	108	191	178
900x135	447	2/890x6	133	2	44	40	10	70	108	472	356
900x180	586	3/890x6	173	2	44	40	10	70	108	649	535
1200x90	522	1180x6	73	2	58	40	14	70	144	340	313
1200x135	757	2/1180x6	133	2	58	40	12	70	140	731	627
1200x180	993	3/1180x6	173	2	58	40	12	70	140	1006	940

North Canterbury Schools, using LVL portals and WS dowels and steel plates

(Above and below) Marshlands School using LVL portals and LVL gussets.

5.0 Further Reading

Design of Epoxied Steel Rods in Glulam Timber. A.H. Buchanan and P.J. Moss. Proceedings, 1999 Pacific Timber Engineering Conference, Rotorua. pp 286-293.

Ductile behaviour and group effect of glued-in steel rods. E. Gehri. Paper 34, Joints with Glued in Rods. Part 2.1 PRO 22 International RILEM Symposium on Joints in Timber Structures, 2001.

Effect of Rod Arrangement on Tensile Strength of Epoxied End Bolts in Glulam. U. Korin, A.H. Buchanan and P.J. Moss, 1999. Proceedings, Pacific Timber Engineering Conference, Rotorua, New Zealand. Vol. 2, pp 217-224.

LVL Portal Frame Design. CHH Woodproducts New Zealand, 8008.

EXPAN Design Guide New Zealand, Timber Portal Frames. P. Quenneville.

From Theory to Reality - 30 Years in Glulam Manufacture. K.A. McIntosh, 1989. Proceedings, Second Pacific Timber Engineering Conference, University of Auckland, Auckland, New Zealand, pp 229-234.

General Building Approval Z-9.1-707 CR421 Purbond

Glued Bolts in Glulam - Proposals for CIB Code. H. Riberholt, 1988. International Council for Building Research Studies and Documentation Working Commission CIB-W18. Timber Structures Meeting Twenty-one, Vancouver, Canada.

Glued-in Bolts. C.J. Johansson, 1995. STEP lecture C14. Timber Engineering STEP 1, Structural Timber Educational Programme.

Glulam Portal Frame Swimming Pool Construction. A.H. Buchanan and M.R. Fletcher, 1989. Proceedings, Second Pacific Timber Engineering Conference, Auckland, New Zealand, Vol 1, pp 245-249.

Moment joints in timber Frames usingglued-in steel rods: experimental investigation of long-term performance. M. Fragiacoma, M. Batchelor, C. Wallington, and A. H. Buchanon, 2010.

Production of Radiata Glulam to Demanding Specifications. B. Walford, 1999. Proceedings, Fifth World Conference on Timber Engineering, Montreux, Switzerland. Vol.1. pp 137-144.

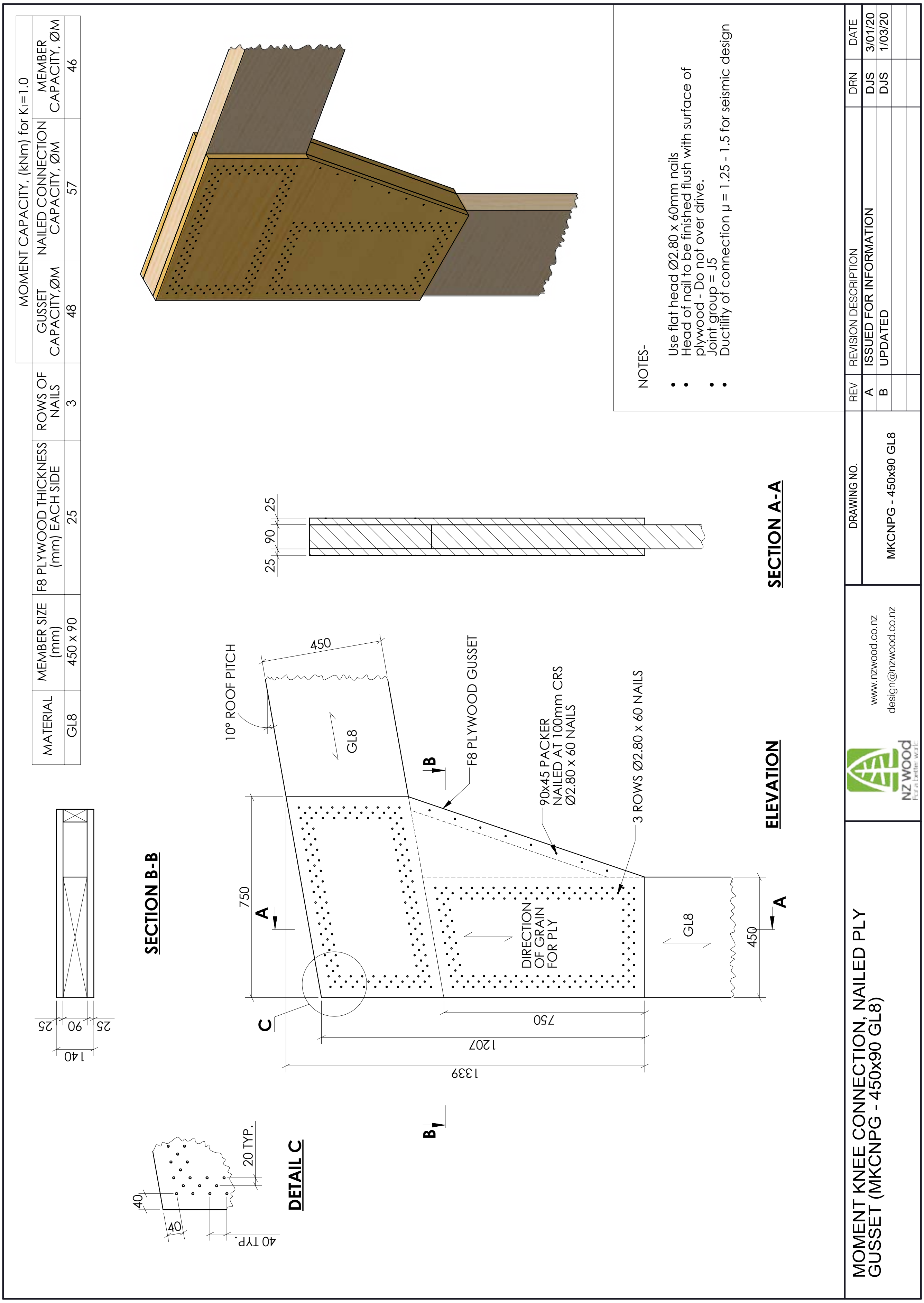
Seismic Design of Glulam Structures. A.H. Buchanan and R.H. Fairweather, 1993. Bulletin of the New Zealand National Society for Earthquake Engineering, 24(4):415-436.

Structural Joints in Glulam. M.L. Batchelar and K.A. McIntosh, 1998. Proceedings, Fifth World Conference on Timber Engineering, Montreux, Switzerland. Vol.1. pp 289-296.

About the Author

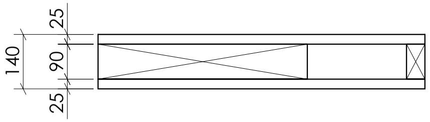
David King is a Structural Engineer and Director of Tasman Consulting Engineers Limited. The company provides structural engineering services to a wide range of clients including residential, light industrial and commercial buildings. David is chairman of the Nelson Timber Solutions Consortium providing specialist structural design for larger scale timber buildings. Civil works include subdivisions, certification for house sites, stormwater and wastewater design.

6.0 Appendix – Example Connections Details



MATERIAL	MEMBER SIZE (mm)	F8 PLYWOOD THICKNESS (mm) EACH SIDE	ROWS OF NAILS	MOMENT CAPACITY, (kNm) for K_1=1.0		
				GUSSET CAPACITY, ØM	NAILED CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
GL8	450 x 90	25	3	48	57	46

MATERIAL	MEMBER SIZE (mm)	F8 PLYWOOD THICKNESS (mm) EACH SIDE	ROWS OF NAILS	MOMENT CAPACITY, (kNm) for K_1=1.0		
				GUSSET CAPACITY,ØM	NAILED CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
GL8	540 x 90	25	3	69	92	66



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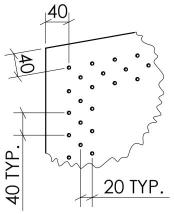

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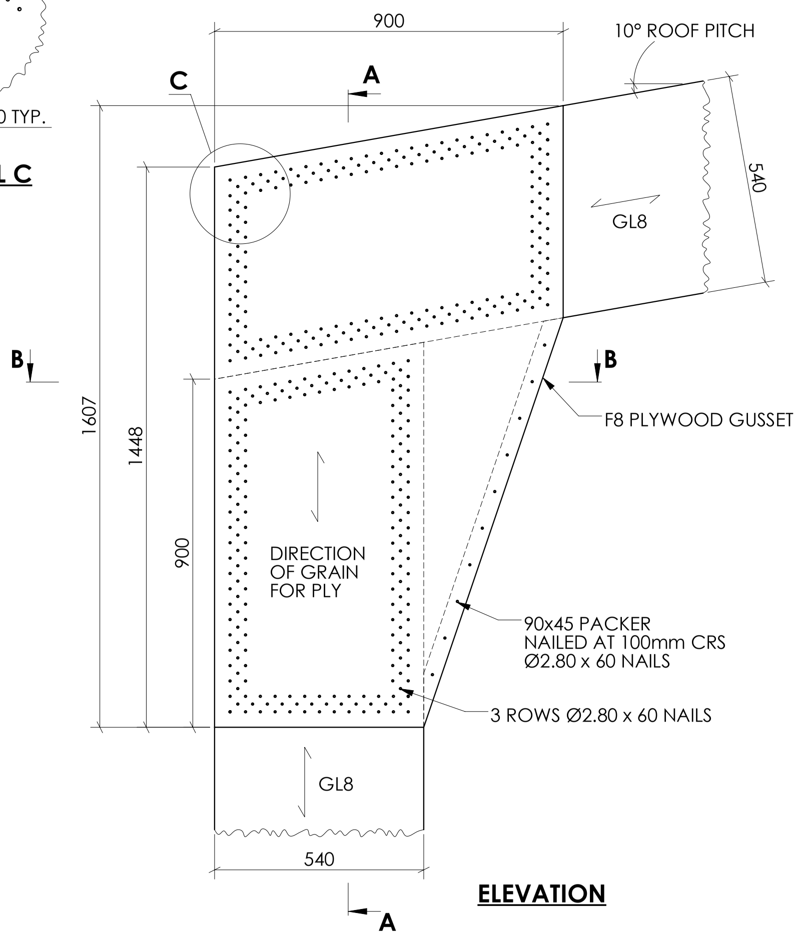

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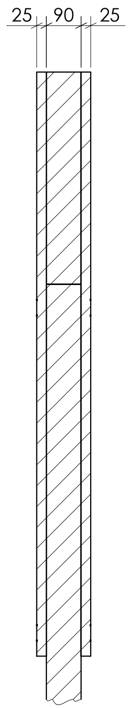

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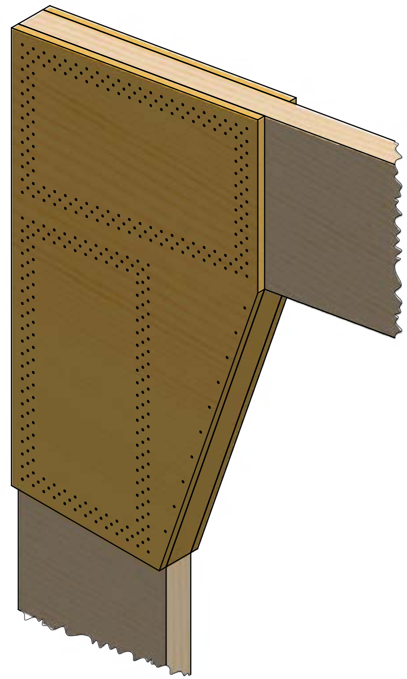


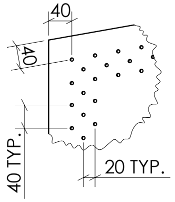
NOTES-

- Use flat head Ø2.80 x 60mm nails
- Head of nail to be finished flush with surface of plywood - Do not over drive.
- Joint group = J5
- Ductility of connection μ = 1.25 - 1.5 for seismic design

MOMENT KNEE CONNECTION, NAILED PLY GUSSET (MKCNPG - 540x90 GL8)	www.nzwood.co.nz design@nzwood.co.nz	DRAWING NO. MKCNPG - 540x90 GL8

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	3/01/20
B	UPDATED	DJS	1/03/20

MATERIAL	MEMBER SIZE (mm)	F8 PLYWOOD THICKNESS (mm) EACH SIDE	ROWS OF NAILS	MOMENT CAPACITY, (kNm) for K_1=1.0		
				GUSSET CAPACITY,ØM	NAILED CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
GL8	630 x 90	25	3	86	122	90



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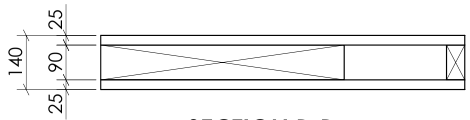

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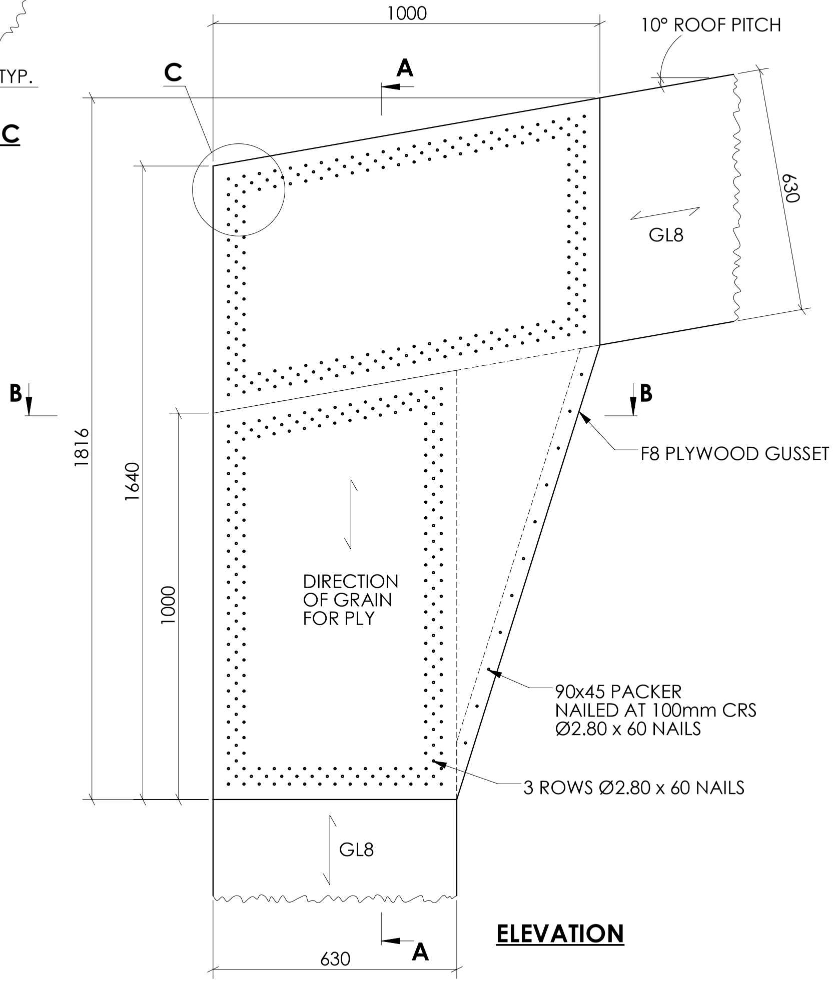

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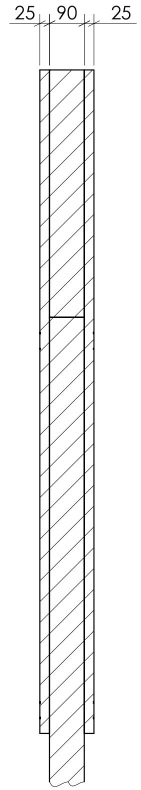

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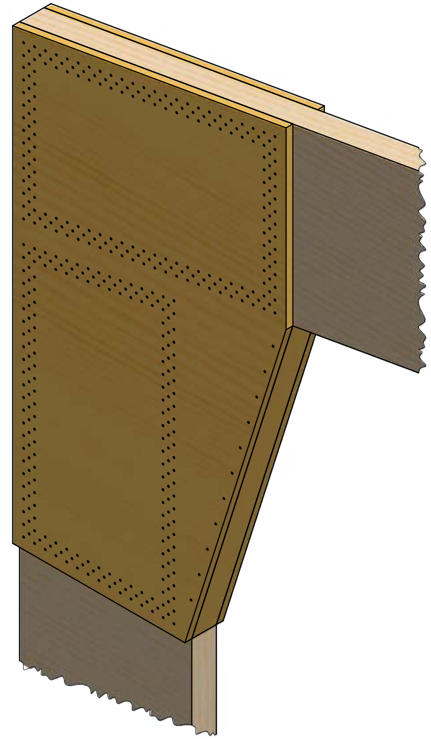

NOTES-

- Use flat head Ø2.80 x 60mm nails
- Head of nail to be finished flush with surface of plywood - Do not over drive.
- Joint group = J5
- Ductility of connection μ = 1.25 - 1.5 for seismic design

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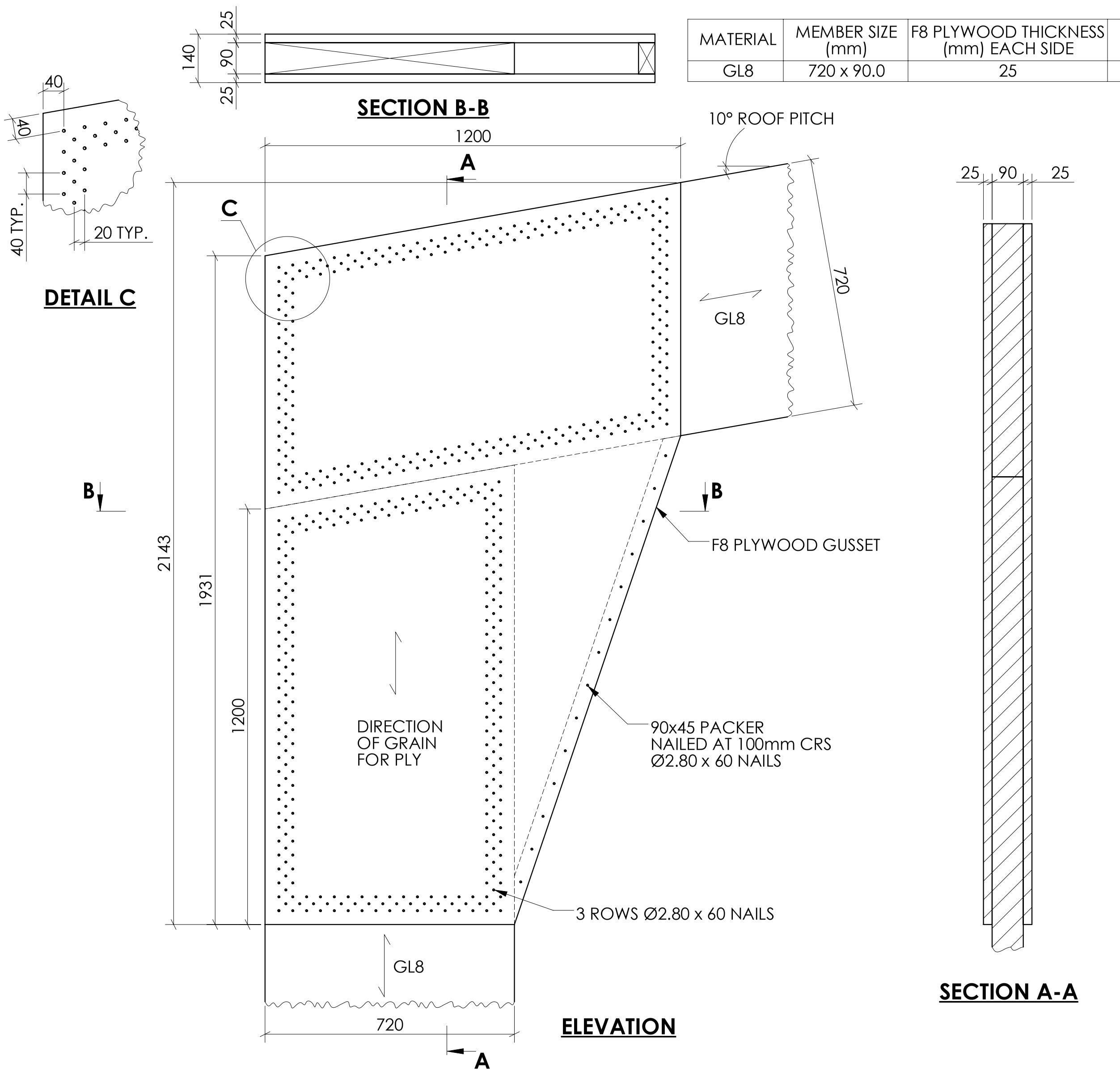
GL8	720 x 90.0	25	3	123	184	118



NOTES-

- Use flat head Ø2.80 x 60mm nails
- Head of nail to be finished flush with surface of plywood - Do not over drive.
- Joint group = J5
- Ductility of connection μ = 1.25 - 1.5 for seismic design

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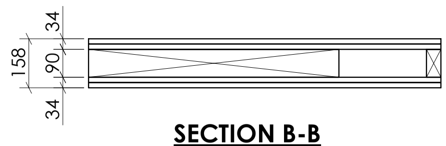
	B	UPDATED	DJS	1/03/20

MATERIAL	MEMBER SIZE (mm)	F8 PLYWOOD THICKNESS (mm) EACH SIDE	ROWS OF NAILS	MOMENT CAPACITY, (kNm) for K_1=1.0		
				GUSSET CAPACITY,ØM	NAILED CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
GL8	810 x 90	2/17	3	180	203	150



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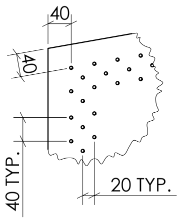

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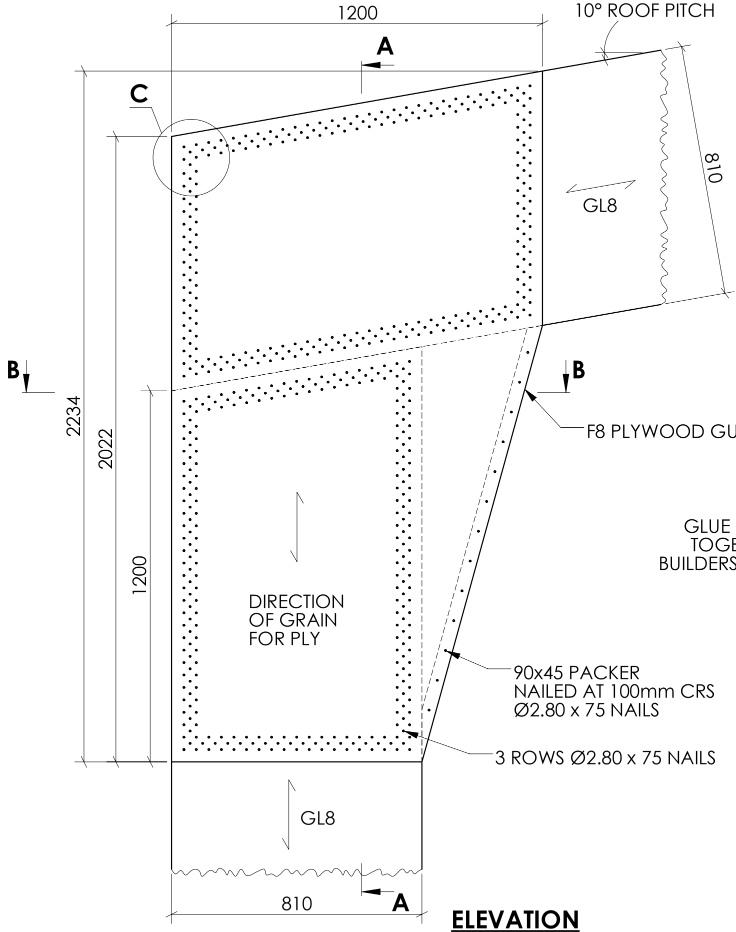

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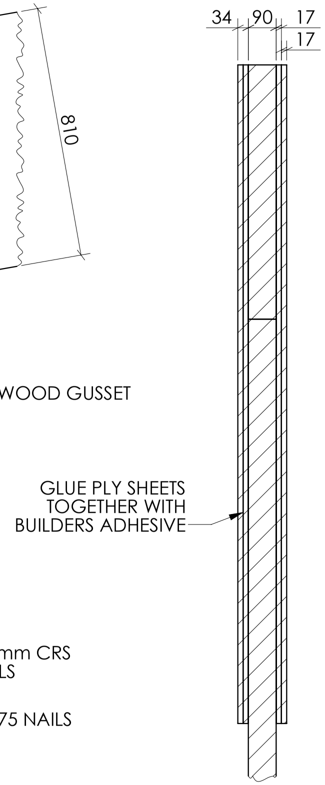

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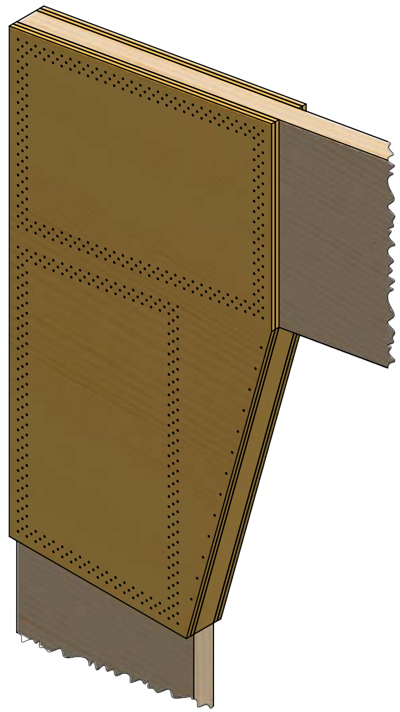

NOTES-

- Use flat head Ø2.80 x 75mm nails
- Head of nail to be finished flush with surface of plywood - Do not over drive.
- Joint group = J5
- Ductility of connection μ = 1.25 - 1.5 for seismic design

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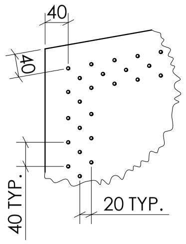
C	UPDATED	DJS	31/03/20



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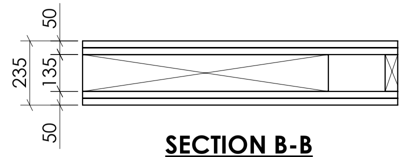
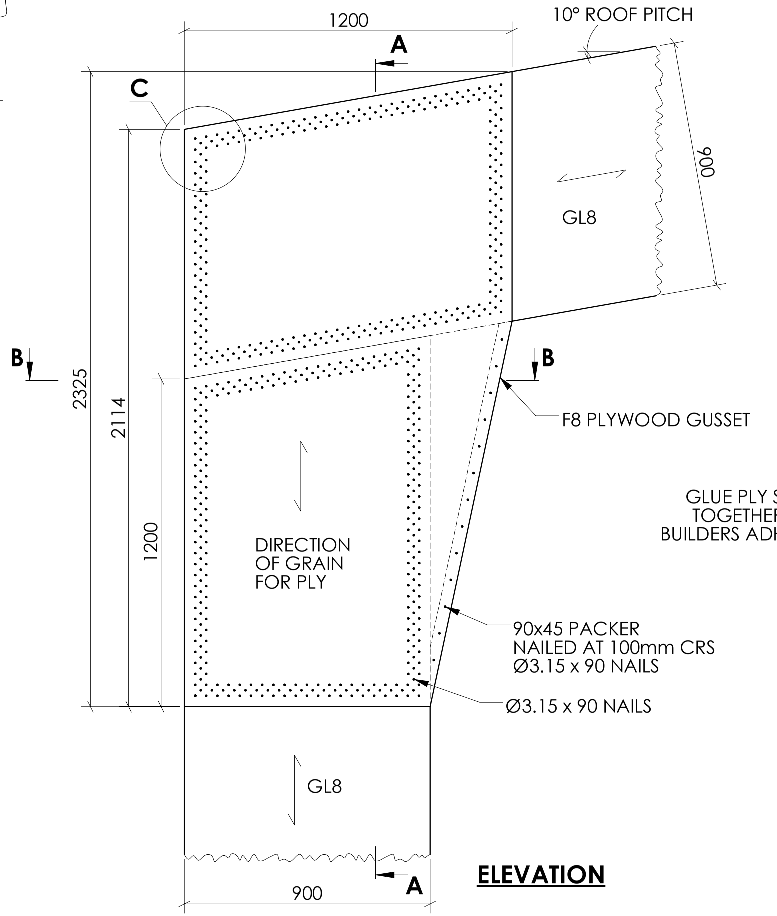


SECTION B-B

MATERIAL	MEMBER SIZE (mm)	F8 PLYWOOD THICKNESS (mm) EACH SIDE	ROWS OF NAILS	MOMENT CAPACITY, (kNm) for K_1=1.0		
				GUSSET CAPACITY,ØM	NAILED CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
GL8	900 x 135	2/25	3	256	281	277



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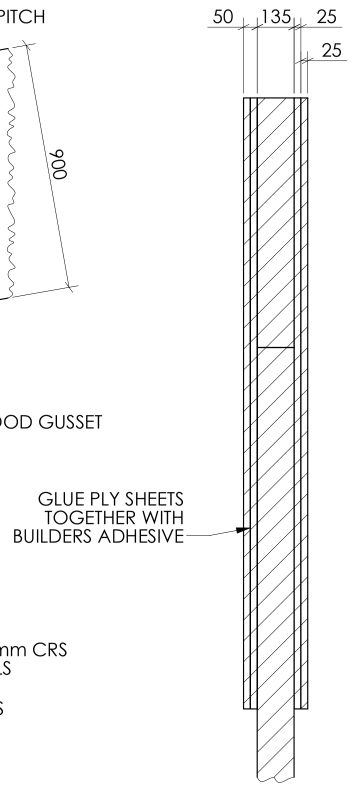

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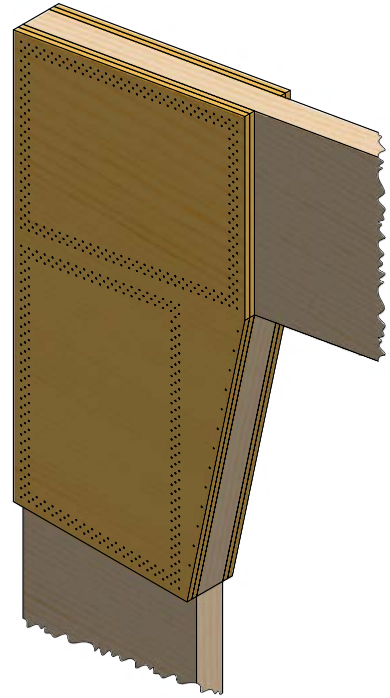

NOTES-

- Use flat head 3.15 x 90mm nails
- Head of nail to be finished flush with surface of plywood - Do not over drive.
- Joint group = J5
- Ductility of connection μ = 1.25 - 1.5 for seismic design

MOMENT KNEE CONNECTION, NAILED PLY GUSSET (MKCNPG - 900x135 GL8)

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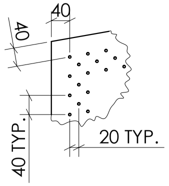
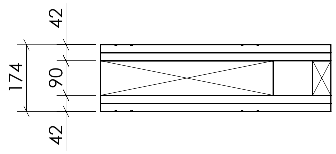
MKCNPG - 900x135 GL8	A	ISSUED FOR INFORMATION	DJS	3/01/20
	B	UPDATED	DJS	1/03/20
	C	UPDATED	DJS	31/03/20

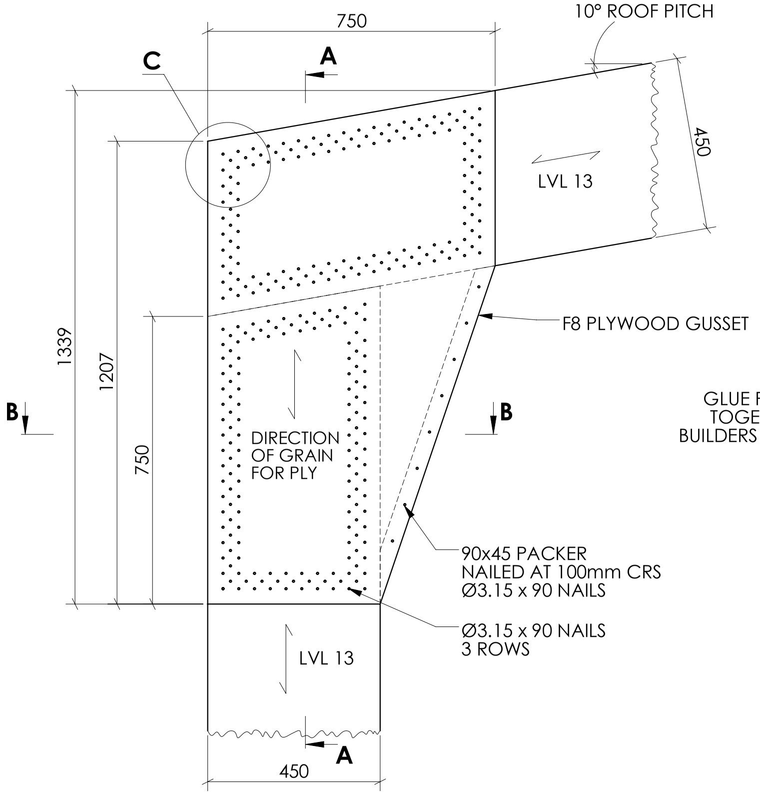
				MOMENT CAPACITY, (kNm) for K_1=1.0		
MATERIAL	MEMBER SIZE (mm)	F8 PLYWOOD THICKNESS (mm) EACH SIDE	ROWS OF NAILS	GUSSET CAPACITY,ØM	NAILED CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
LVL 13	450 x 90	2/21	3	83	71	95



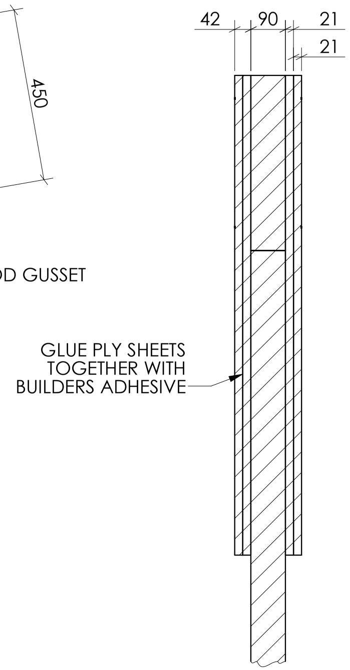
DETAIL C



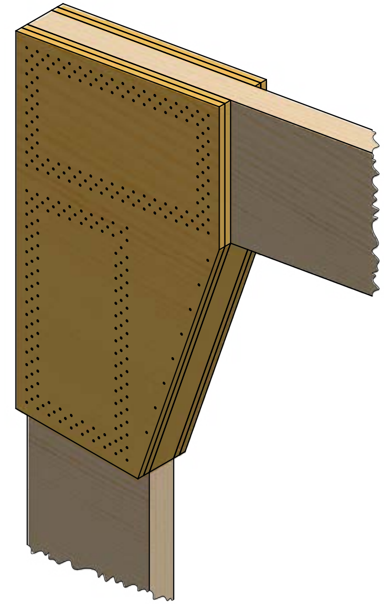
SECTION B-B



ELEVATION



SECTION A-A



NOTES-

- Use flat head Ø3.15 x 90mm nails
- Head of nail to be finished flush with surface of plywood - Do not over drive.
- Joint group = J5
- Ductility of connection μ = 1.25 - 1.5 for seismic design

MOMENT KNEE CONNECTION, NAILED PLY GUSSET (MKCNPG - 450x90 LVL13)

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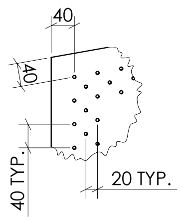
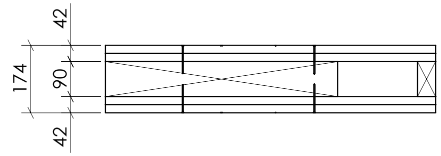
DRAWING NO.	REV	REVISION DESCRIPTION	DRN	DATE
MKCNPG - 450x90 LVL13	B	UPDATED	DJS	3/05/19
	C	UPDATED	DJS	3/01/20
	D	UPDATED	DJS	23/02/20
	E	UPDATED	DJS	31/03/20

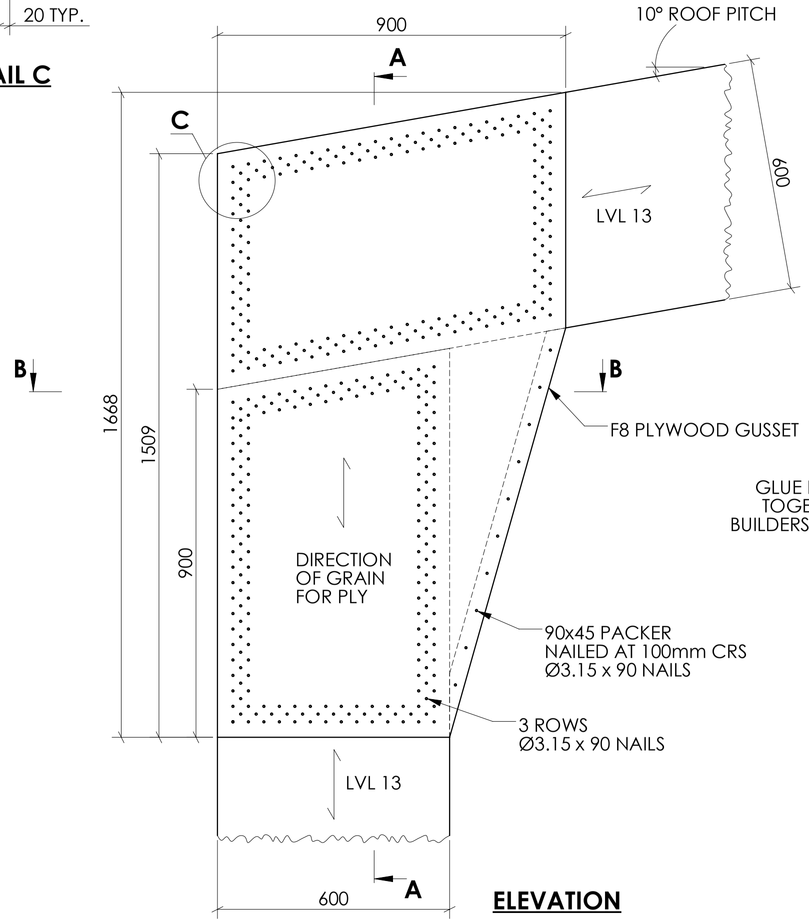
MATERIAL	MEMBER SIZE (mm)	F8 PLYWOOD THICKNESS (mm) EACH SIDE	ROWS OF NAILS	MOMENT CAPACITY, (kNm) for K_1=1.0		
				GUSSET CAPACITY,ØM	NAILED CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
LVL 13	600 x 90	2/21	3	120	125	161



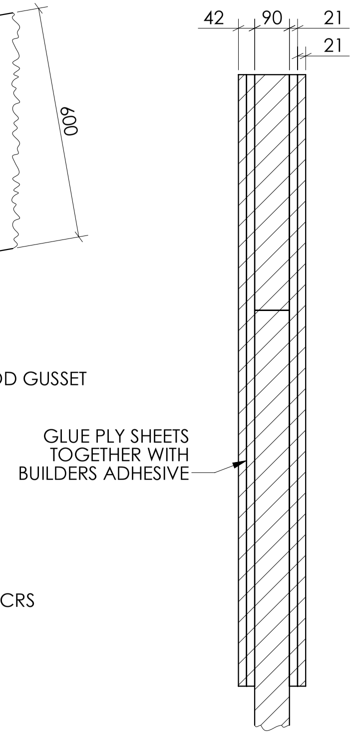
DETAIL C



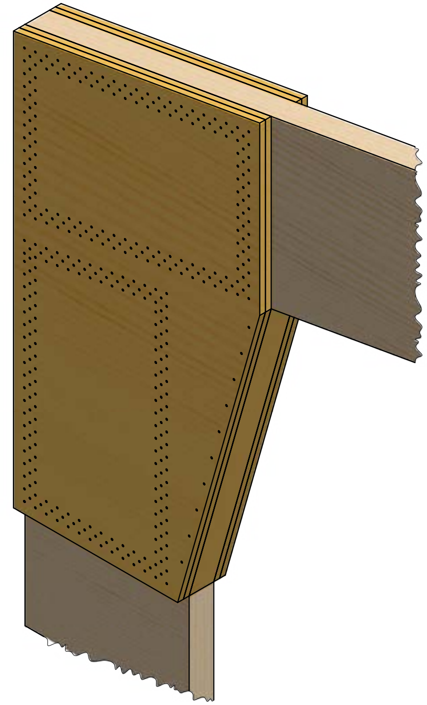
SECTION B-B



ELEVATION



SECTION A-A



NOTES-

- Use flat head 3.15 x 90mm nails
- Head of nail to be finished flush with surface of plywood - Do not over drive.
- Joint group = J5
- Ductility of connection μ = 1.25 - 1.5 for seismic design

MOMENT KNEE CONNECTION, NAILED PLY GUSSET (MKCNPG - 600x90 LVL13)

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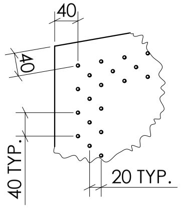
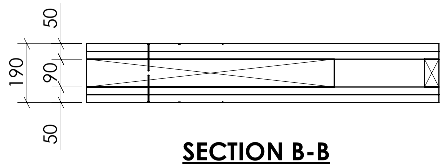
DRAWING NO.
MKCNPG - 600x90 LVL13

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	7/03/19
B	UPDATED	DJS	3/01/20
C	UPDATED	DJS	23/02/20
D	UPDATED	DJS	31/03/20

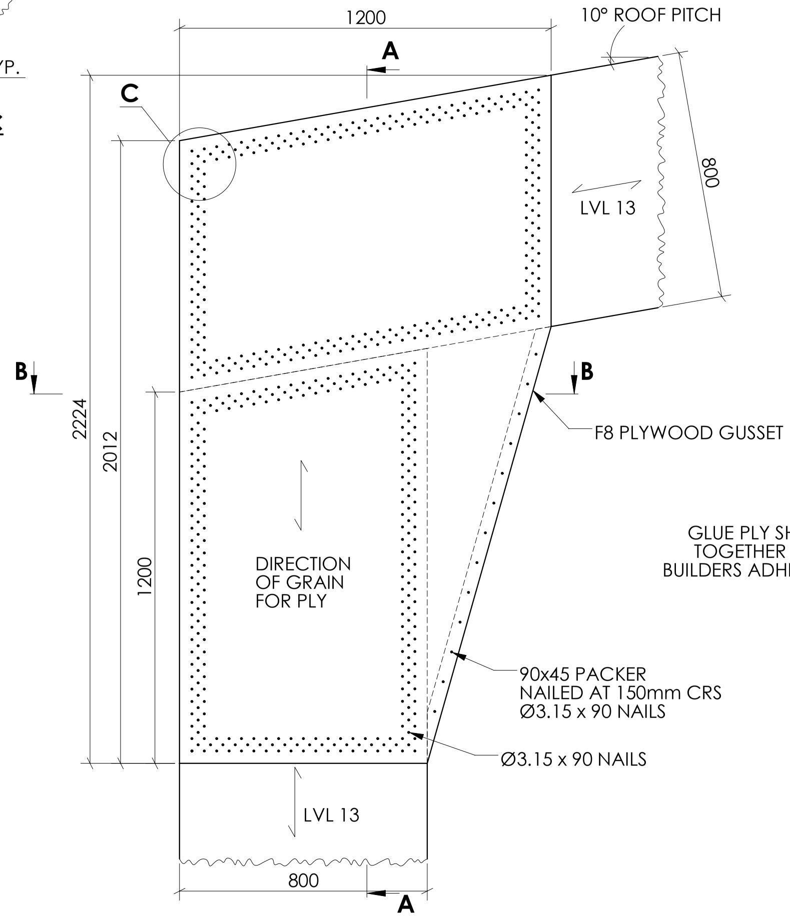
MATERIAL	MEMBER SIZE (mm)	F8 PLYWOOD THICKNESS (mm) EACH SIDE	ROWS OF NAILS	MOMENT CAPACITY, (kNm) for K_1=1.0 GUSSET CAPACITY,ØM	NAILED CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
LVL 13	800 x 90	2/25	3	250	251	272



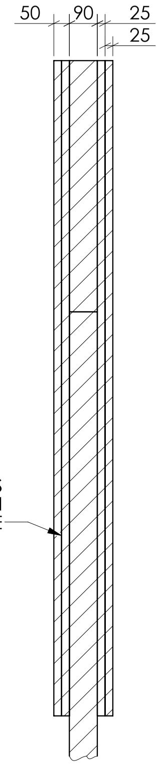
DETAIL C



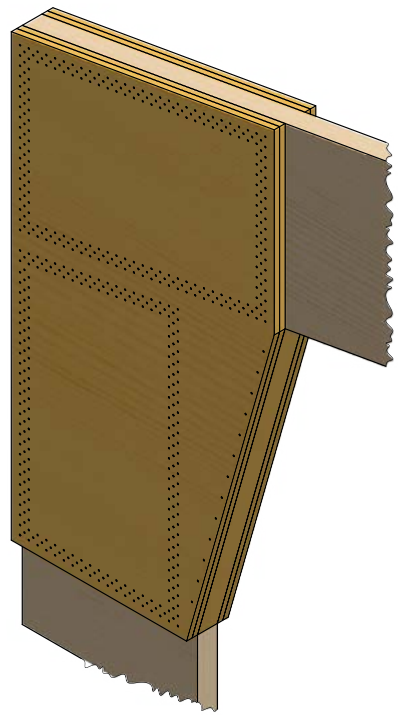
SECTION B-B



ELEVATION



SECTION A-A



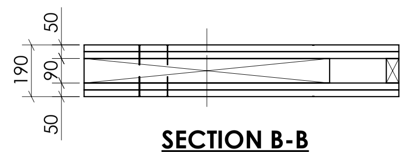
NOTES-

- Use flat head 3.15 x 90mm nails
- Head of nail to be finished flush with surface of plywood - Do not over drive.
- Joint group = J5
- Ductility of connection μ = 1.25 - 1.5 for seismic design

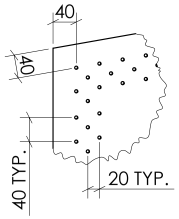
MOMENT KNEE CONNECTION, NAILED PLY GUSSET (MKCNPG - 800x90 LVL13)	NZ wood For a better world www.nzwood.co.nz design@nzwood.co.nz	DRAWING NO. MKCNPG - 800x90 LVL13

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	7/03/19
B	UPDATED	DJS	3/01/20
C	UPDATED	DJS	27/02/20
D	UPDATED	DJS	31/03/20

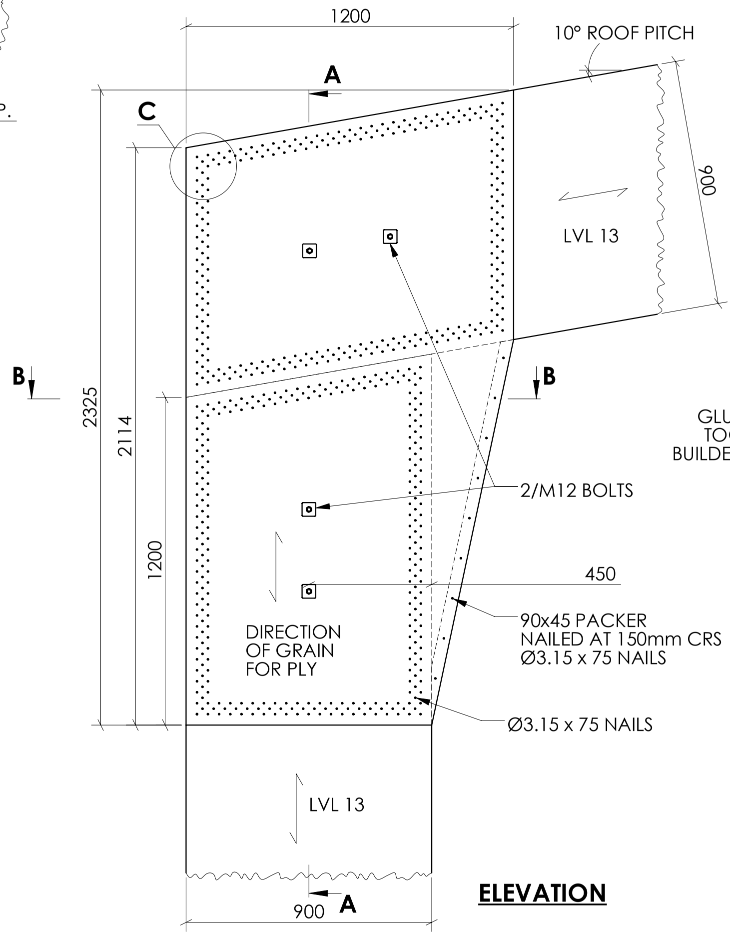
MATERIAL	MEMBER SIZE (mm)	F11 PLYWOOD THICKNESS (mm) EACH SIDE	ROWS OF NAILS	MOMENT CAPACITY, (kNm) for K_1=1.0 GUSSET CAPACITY,ØM	NAILED CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
LVL 13	900 x 90	2/25	3	328	281	338



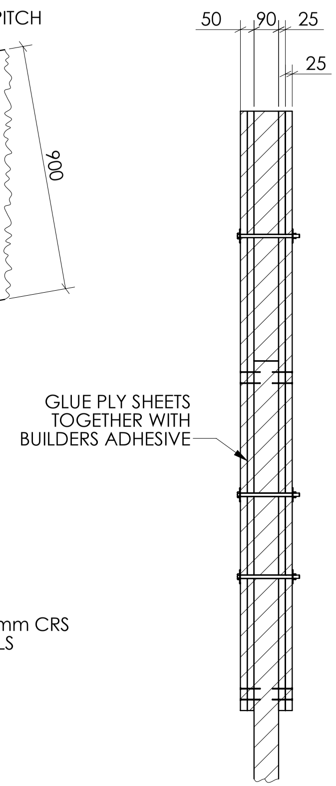
SECTION B-B



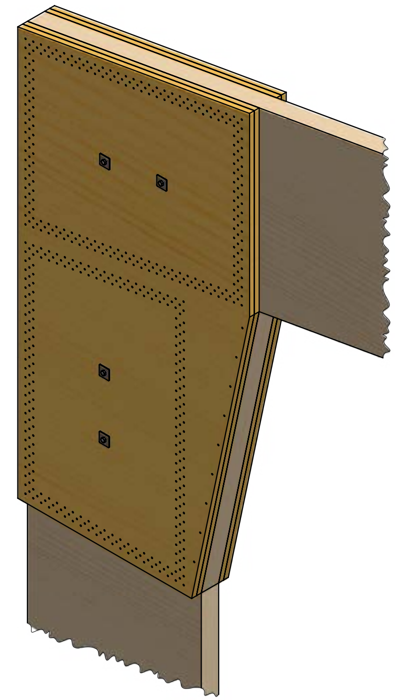
DETAIL C



ELEVATION



SECTION A-A



NOTES-

- Use flat head 3.15 x 90mm nails
- Head of nail to be finished flush with surface of plywood - Do not over drive.
- Joint group = J5
- Ductility of connection μ = 1.25 - 1.5 for seismic design

MOMENT KNEE CONNECTION, NAILED PLY GUSSET (MKCNPG - 900x90 LVL13)

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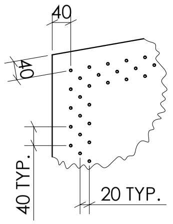
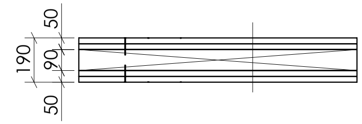
DRAWING NO.	REV	REVISION DESCRIPTION	DRN	DATE
MKCNPG - 900x90 LVL13	A	ISSUED FOR INFORMATION	DJS	7/03/19
	B	UPDATED	DJS	3/01/20
	C	UPDATED	DJS	28/02/20
	D	UPDATED	DJS	31/03/20

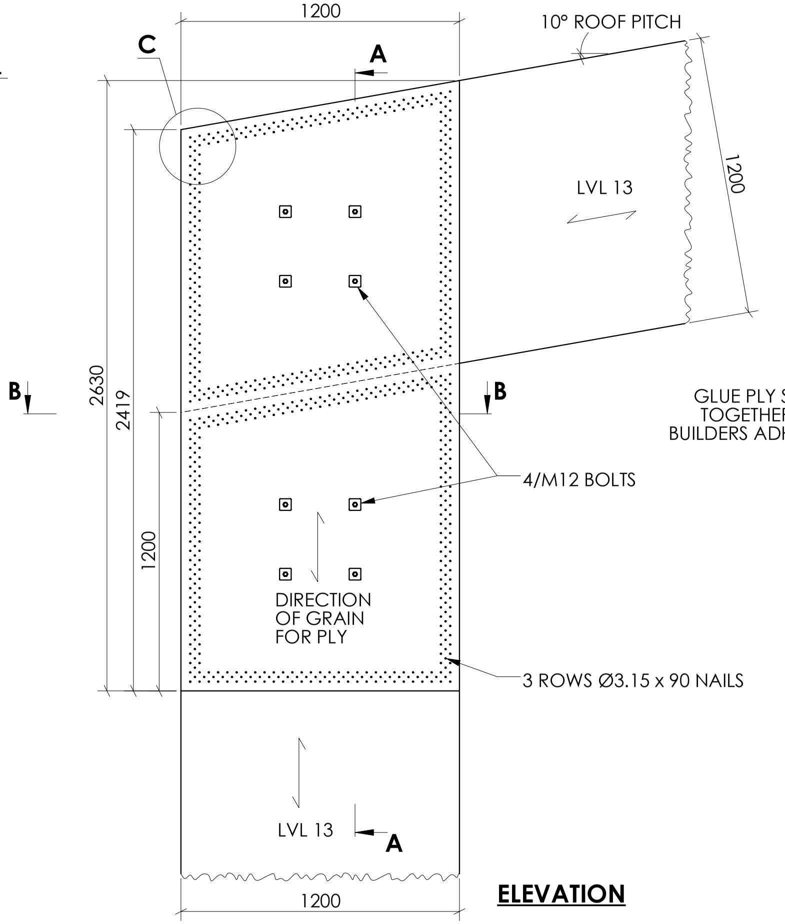
				MOMENT CAPACITY, (kNm) for K_1=1.0		
MATERIAL	MEMBER SIZE (mm)	F11 PLYWOOD THICKNESS (mm) EACH SIDE	ROWS OF NAILS	GUSSET CAPACITY,ØM	NAILED CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
LVL 13	1200 x 90	2/25	3	345	380	573



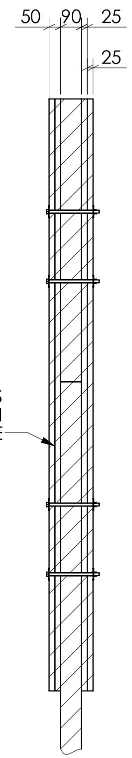
DETAIL C



SECTION B-B



ELEVATION



SECTION A-A

NOTES-

- Use flat head 3.15 x 90mm nails
- Head of nail to be finished flush with surface of plywood - Do not over drive.
- Joint group = J5
- Ductility of connection μ = 1.25 - 1.5 for seismic design

MOMENT KNEE CONNECTION, NAILED PLY GUSSET (MKCNPG - 1200x90 LVL13)

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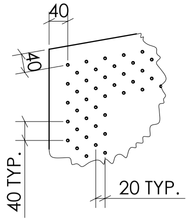
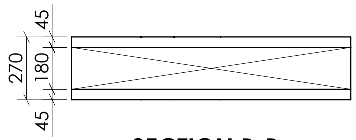
DRAWING NO.
MKCNPG - 1200x90 LVL13

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	7/03/19
B	UPDATED	DJS	3/01/20
C	UPDATED	DJS	1/03/20
D	UPDATED	DJS	31/03/20

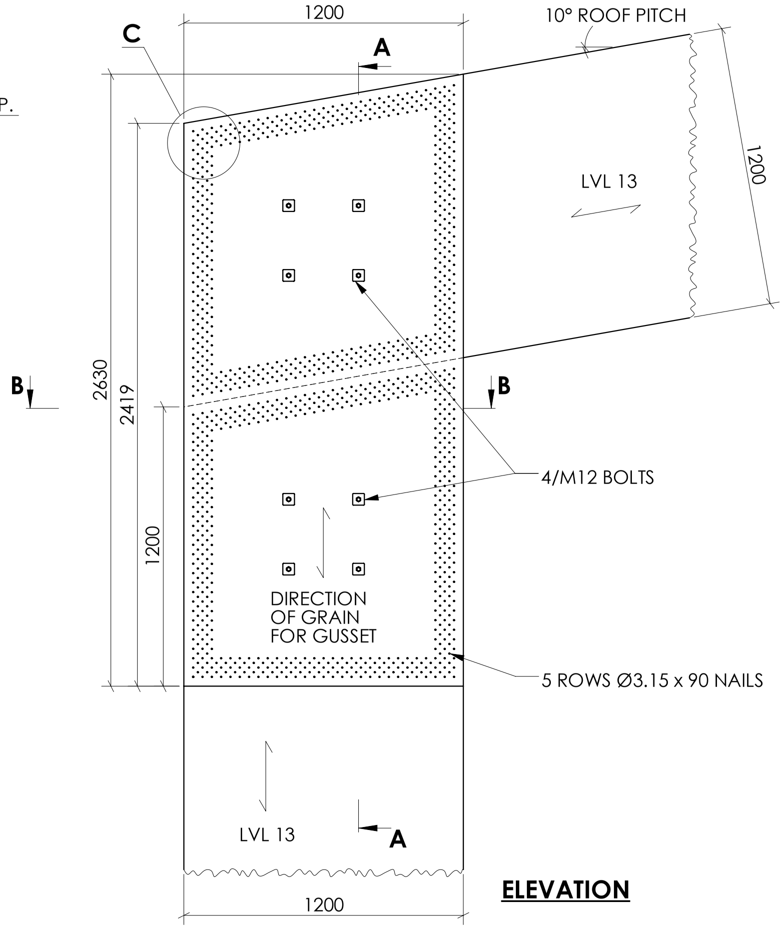
MATERIAL	MEMBER SIZE (mm)	LVL13 THICKNESS (mm) EACH SIDE	ROWS OF NAILS	MOMENT CAPACITY, (kNm) for K_1=1.0 GUSSET CAPACITY,ØM	NAILED CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
LVL 13	1200 x 180	45	5	573	566	1146



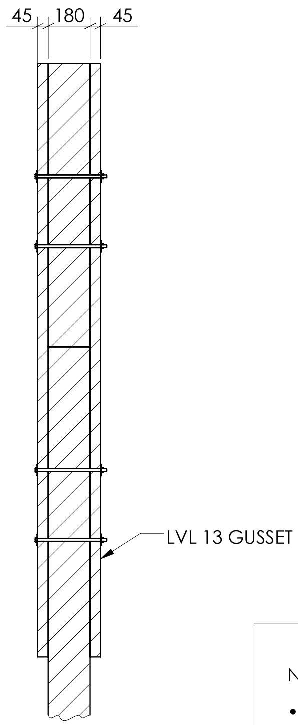
DETAIL C



SECTION B-B



ELEVATION



SECTION A-A

NOTES-

- Use flat head 3.15 x 90mm nails
- Head of nail to be finished flush with surface of plywood - Do not over drive.
- Joint group = J5
- Ductility of connection μ = 1.25 - 1.5 for seismic design

MOMENT KNEE CONNECTION, NAILED PLY GUSSET (MKCNPG - 1200x180 LVL13)

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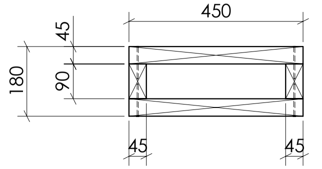
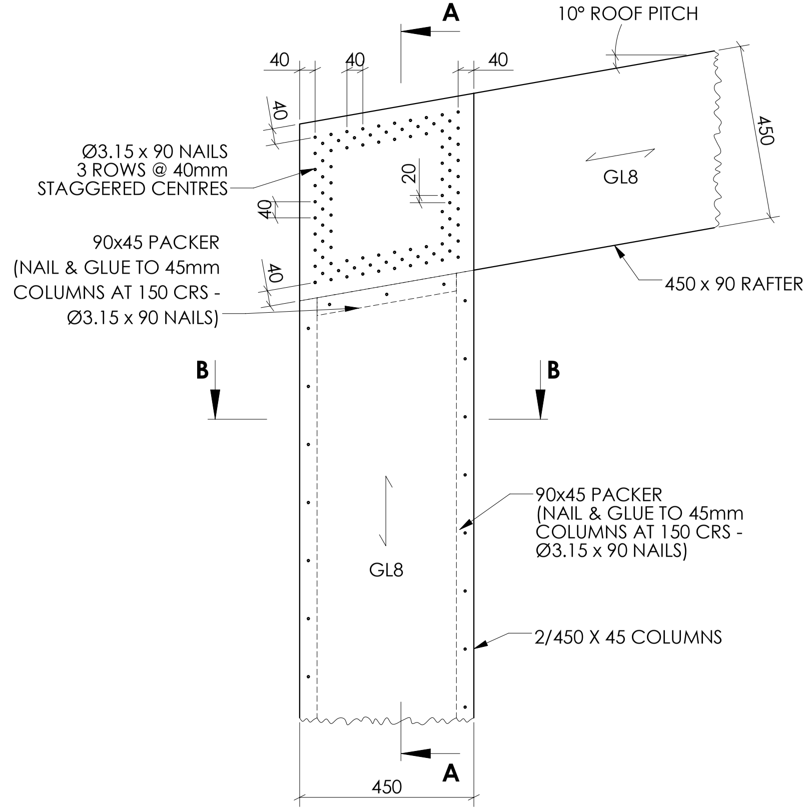
DRAWING NO. MKCNPG - 1200x180 LVL13

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	7/03/19
B	UPDATED	DJS	3/01/20
C	UPDATED	DJS	1/03/20

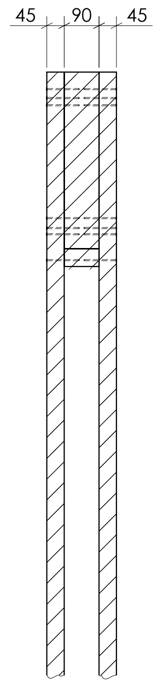


SECTION B-B

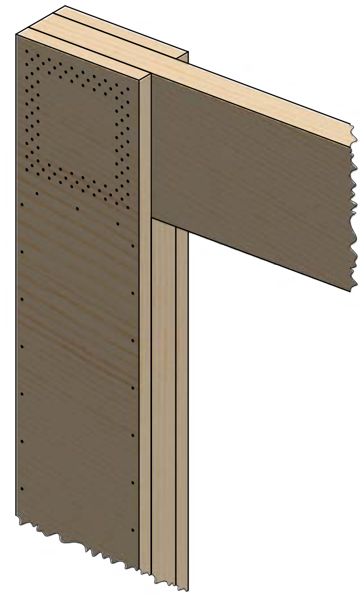
MATERIAL	RAFTER SIZE (mm)	COLUMN SIZE (mm)	ROWS OF NAILS	MOMENT CAPACITY, (kNm) for K_1=1.0 NAILED CONNECTION CAPACITY, ØM	MOMENT CAPACITY, (kNm) for K_1=1.0 MEMBER CAPACITY, ØM
GL8	450 x 90	2/450x45	3	32	46



ELEVATION



SECTION A-A



NOTES-

- Use flat head Ø3.15 x 90mm nails
- Head of nail to be finished flush with surface - do not over drive.
- Joint group = J5
- Ductility of connection μ=1.25-1.5 for seismic design

MOMENT KNEE CONNECTION, LAP JOINT
(MKCLJ - 450x90 GL8)

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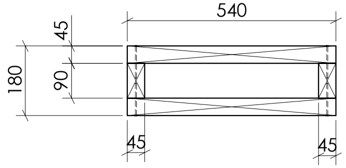
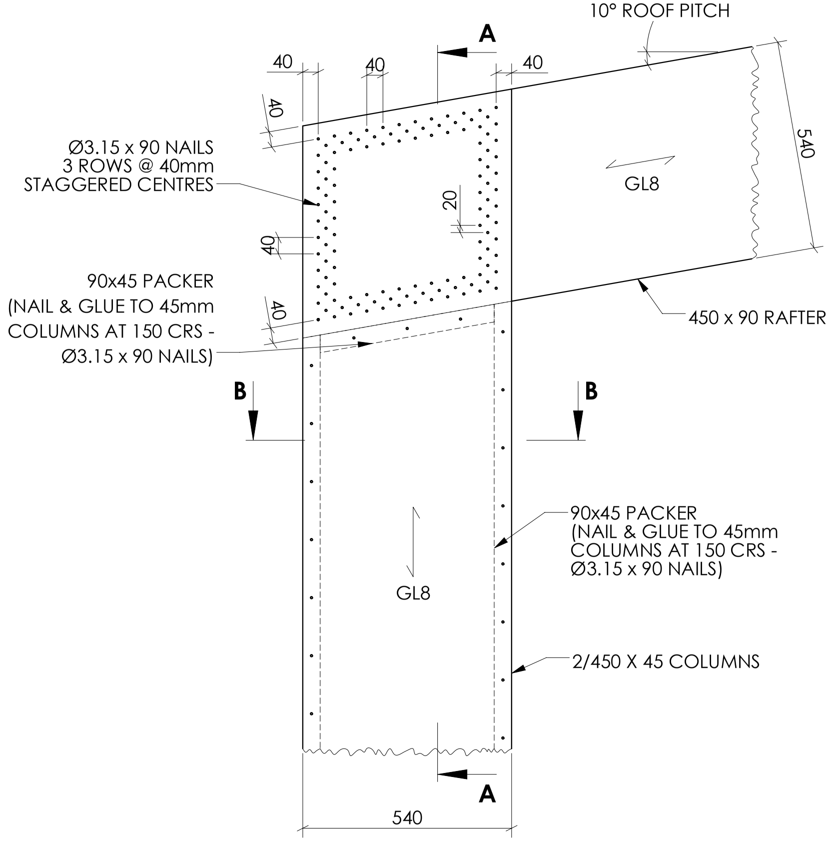
DRAWING NO.
MKCLJ - 450x90 GL8

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	3/06/19
B	UPDATED	DJS	30/12/19
C	UPDATED	DJS	23/02/20

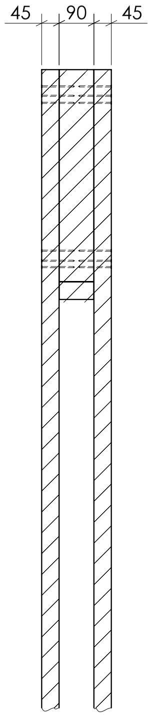
MATERIAL	RAFTER SIZE (mm)	COLUMN SIZE (mm)	ROWS OF NAILS	MOMENT CAPACITY, (kNm) for K_1=1.0 NAILED CONNECTION CAPACITY, ØM	MOMENT CAPACITY, (kNm) for K_1=1.0 MEMBER CAPACITY, ØM
GL8	540 x 90	2/540x45	3	52	66



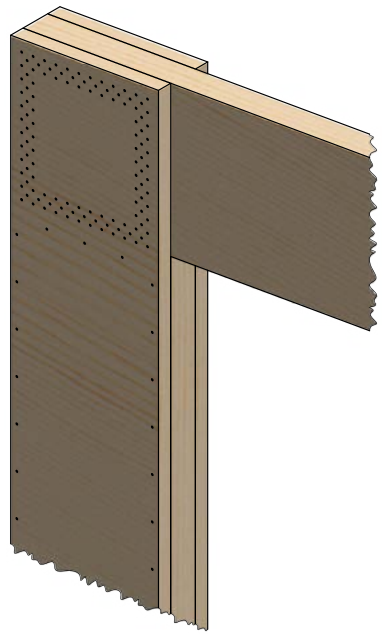
SECTION B-B



ELEVATION



SECTION A-A



NOTES-

- Use flat head Ø3.15 x 90mm nails
- Head of nail to be finished flush with surface - do not over drive.
- Joint group = J5
- Ductility of connection μ=1.25-1.5 for seismic design

MOMENT KNEE CONNECTION, LAP JOINT
(MKCLJ - 540x90 GL8)

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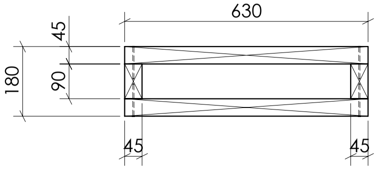
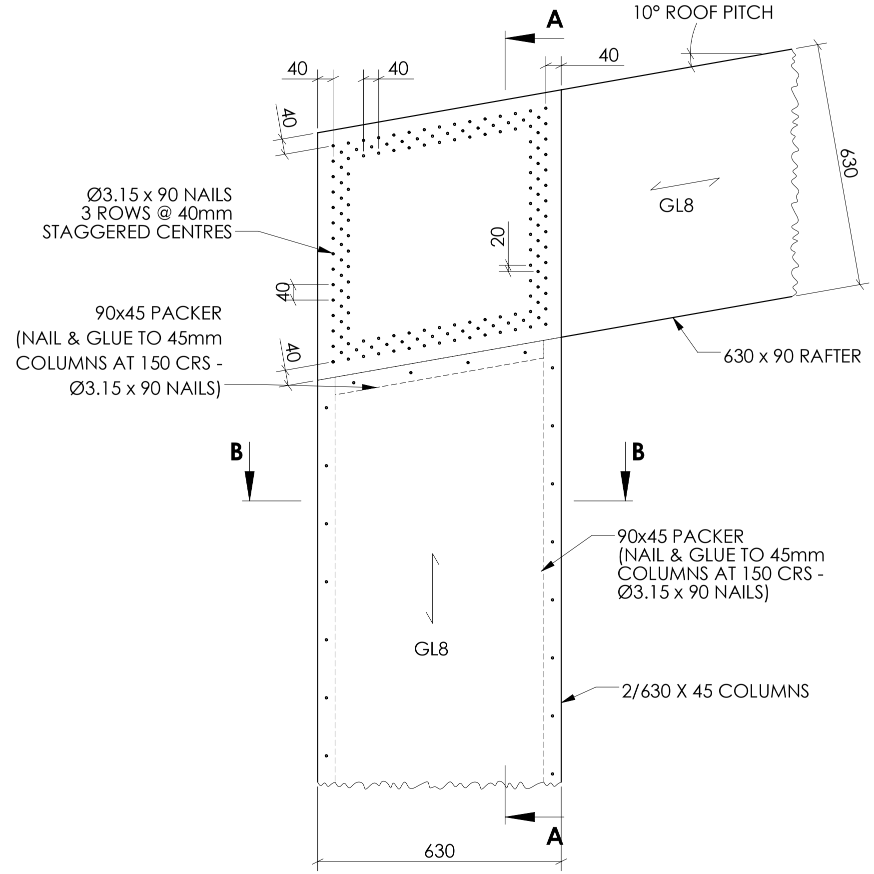
DRAWING NO.
MKCLJ - 540x90 GL8

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	3/06/19
B	UPDATED	DJS	30/12/19
C	UPDATED	DJS	23/02/20

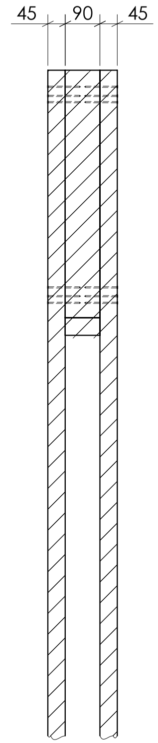
MATERIAL	RAFTER SIZE (mm)	COLUMN SIZE (mm)	ROWS OF NAILS	MOMENT CAPACITY, (kNm) for K_1=1.0 NAILED CONNECTION CAPACITY, ØM	MOMENT CAPACITY, (kNm) for K_1=1.0 MEMBER CAPACITY, ØM
GL8	630 x 90	2/630x45	3	79	90



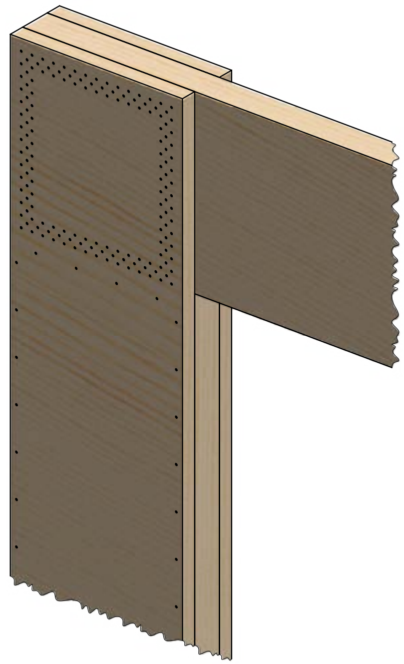
SECTION B-B



ELEVATION



SECTION A-A



NOTES-

- Use flat head Ø3.15 x 90mm nails
- Head of nail to be finished flush with surface - do not over drive.
- Joint group = J5
- Ductility of connection μ=1.25-1.5 for seismic design

MOMENT KNEE CONNECTION, LAP JOINT
(MKCLJ - 630x90 GL8)

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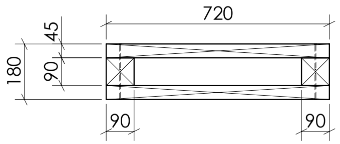
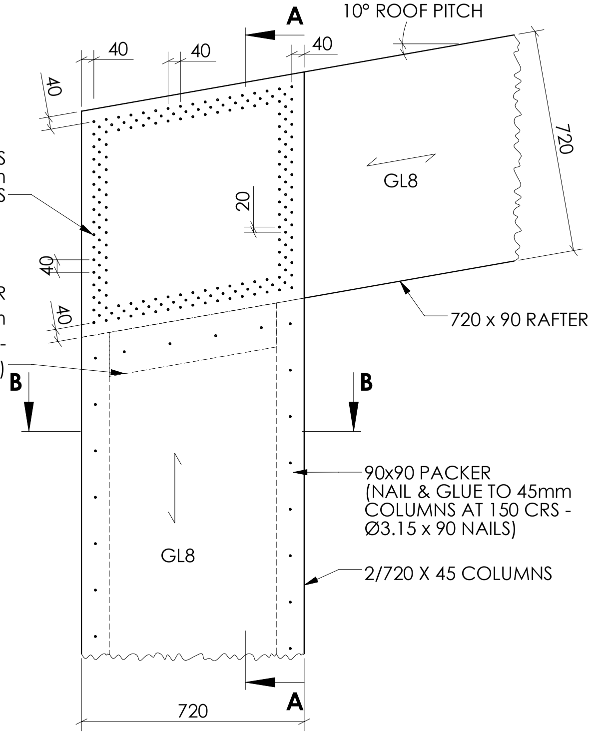
DRAWING NO.
MKCLJ - 630x90 GL8

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	3/06/19
B	UPDATED	DJS	30/12/19
C	UPDATED	DJS	23/02/20

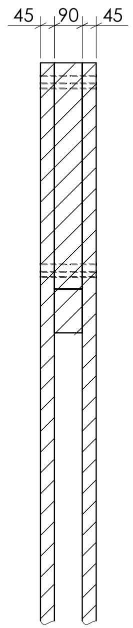
MATERIAL	RAFTER SIZE (mm)	COLUMN SIZE (mm)	ROWS OF NAILS	MOMENT CAPACITY, (kNm) for K_1=1.0 NAILED CONNECTION CAPACITY, ØM	MOMENT CAPACITY, (kNm) for K_1=1.0 MEMBER CAPACITY, ØM
GL8	720 x 90	2/720x45	3	111	118



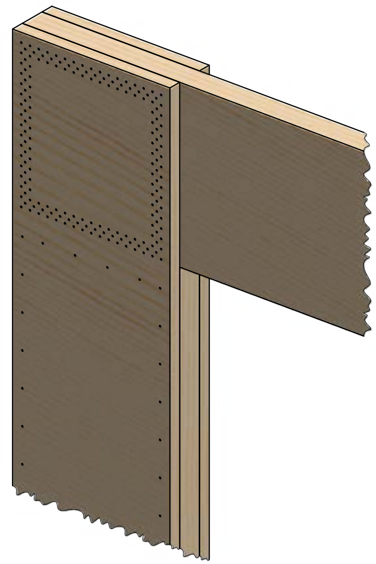
SECTION B-B



ELEVATION



SECTION A-A



NOTES-

- Use flat head Ø3.15 x 90mm nails
- Head of nail to be finished flush with surface - do not over drive.
- Joint group = J5
- Ductility of connection μ=1.25-1.5 for seismic design

MOMENT KNEE CONNECTION, LAP JOINT
(MKCLJ - 720x90 GL8)

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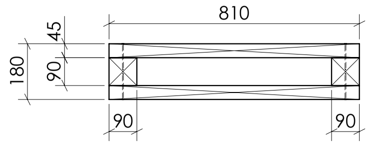
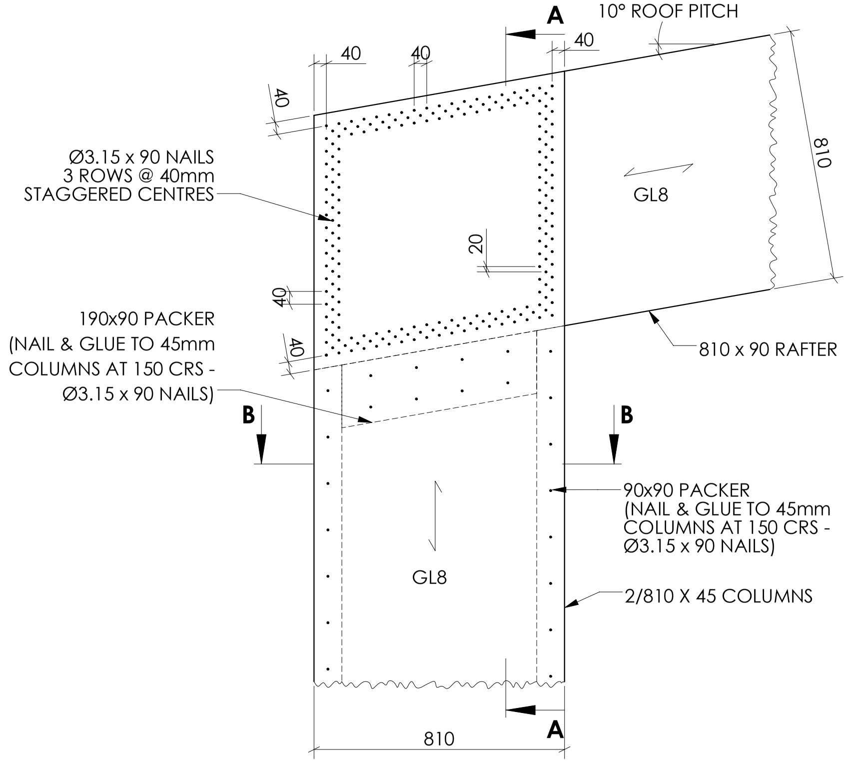
DRAWING NO.
MKCLJ - 720x90 GL8

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	3/06/19
B	UPDATED	DJS	30/12/19
C	UPDATED	DJS	23/02/20

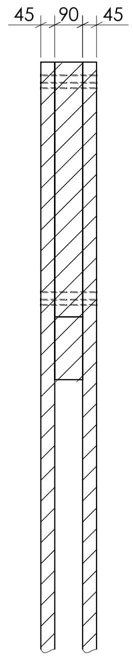


SECTION B-B

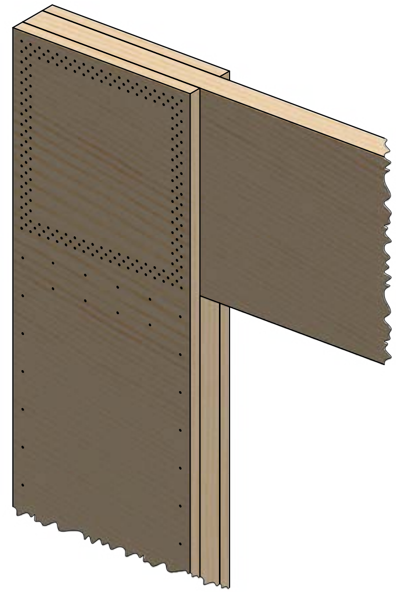
MATERIAL	RAFTER SIZE (mm)	COLUMN SIZE (mm)	ROWS OF NAILS	MOMENT CAPACITY, (kNm) for K_1=1.0 NAILED CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
GL8	810 x 90	2/810x45	3	150	150



ELEVATION



SECTION A-A



NOTES-

- Use flat head Ø3.15 x 90mm nails
- Head of nail to be finished flush with surface - do not over drive.
- Joint group = J5
- Ductility of connection μ=1.25-1.5 for seismic design

MOMENT KNEE CONNECTION, LAP JOINT
(MKCLJ - 810x90 GL8)

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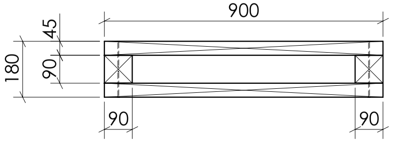
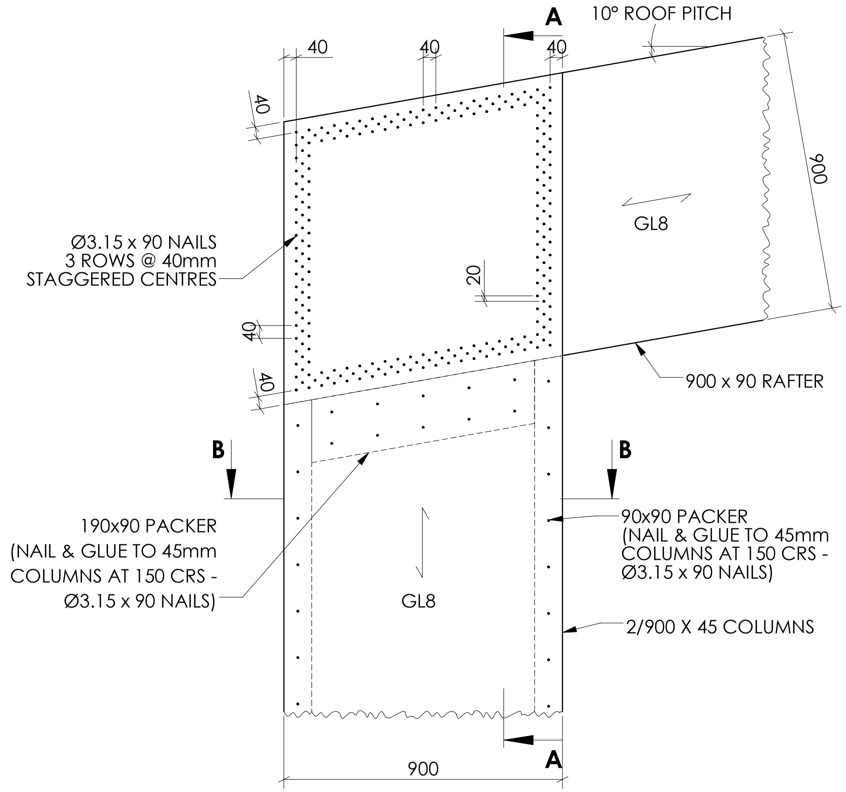
DRAWING NO.	REV	REVISION DESCRIPTION	DRN	DATE
MKCLJ - 810x90 GL8	A	ISSUED FOR INFORMATION	DJS	3/06/19
	B	UPDATED	DJS	30/12/19
	C	UPDATED	DJS	23/02/20

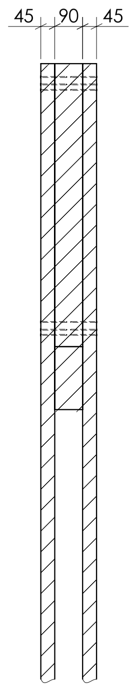
MATERIAL	RAFTER SIZE (mm)	COLUMN SIZE (mm)	ROWS OF NAILS	MOMENT CAPACITY, (kNm) for K_1=1.0 NAILED CONNECTION CAPACITY, ØM	MOMENT CAPACITY, (kNm) for K_1=1.0 MEMBER CAPACITY, ØM
GL8	900 x 90	2/900x45	3	194	185



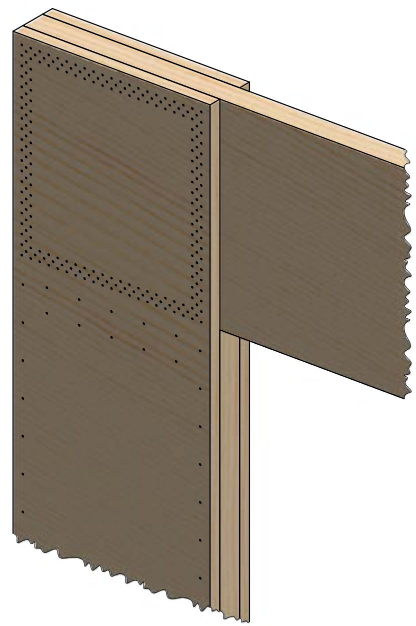
SECTION B-B



ELEVATION



SECTION A-A



NOTES-

- Use flat head Ø3.15 x 90mm nails
- Head of nail to be finished flush with surface - do not over drive.
- Joint group = J5
- Ductility of connection μ=1.25-1.5 for seismic design

MOMENT KNEE CONNECTION, LAP JOINT
(MKCLJ - 900x90 GL8)

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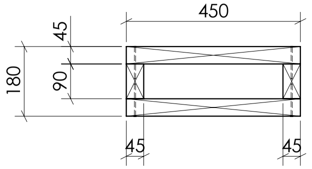
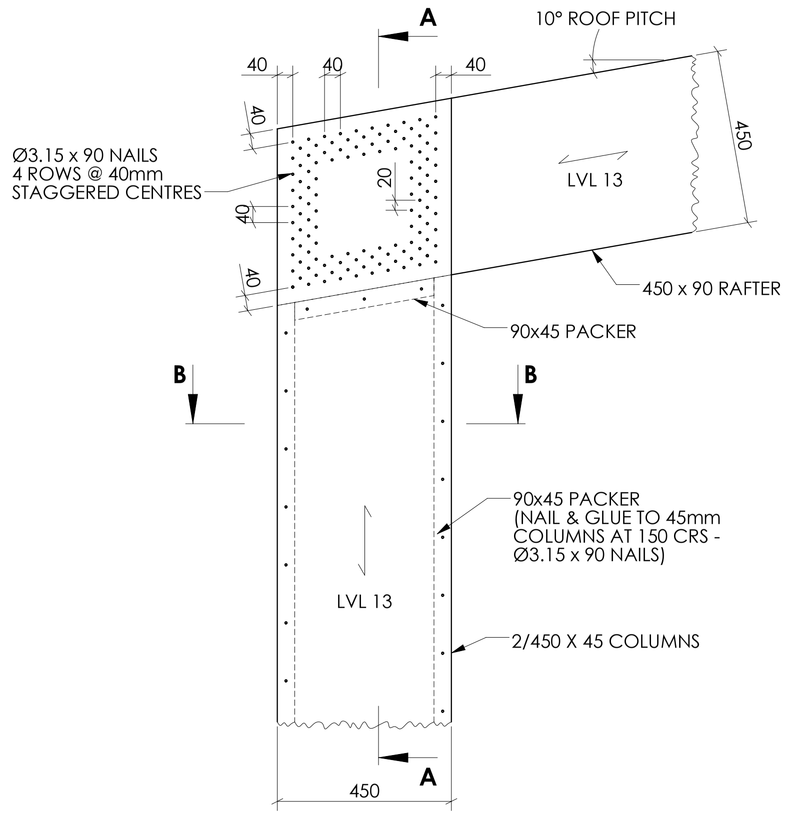
DRAWING NO.
MKCLJ - 900x90 GL8

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	3/06/19
B	UPDATED	DJS	30/12/19
C	UPDATED	DJS	23/02/20

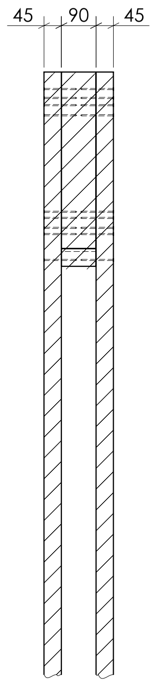
MATERIAL	RAFTER SIZE (mm)	COLUMN SIZE (mm)	ROWS OF NAILS	MOMENT CAPACITY, (kNm) for K_1=1.0	
				NAILED CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
LVL 13	450 x 90	2/450x45	4	34	95



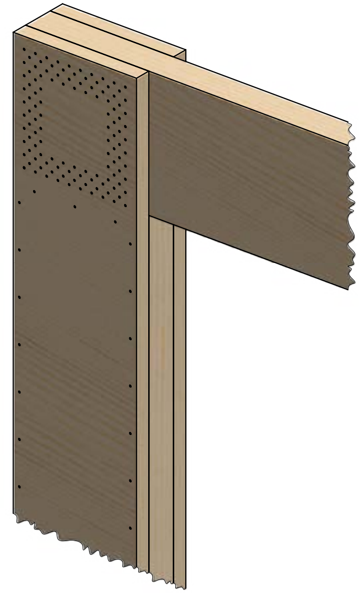
SECTION B-B



ELEVATION



SECTION A-A



NOTES-

- Use flat head Ø3.15 x 90mm nails
- Head of nail to be finished flush with surface - do not over drive.
- Joint group = J5
- Ductility of connection μ=1.25-1.5 for seismic design

MOMENT KNEE CONNECTION, LAP JOINT
(MKCLJ - 450x90 LVL13)

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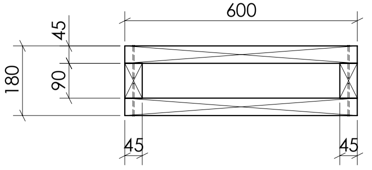
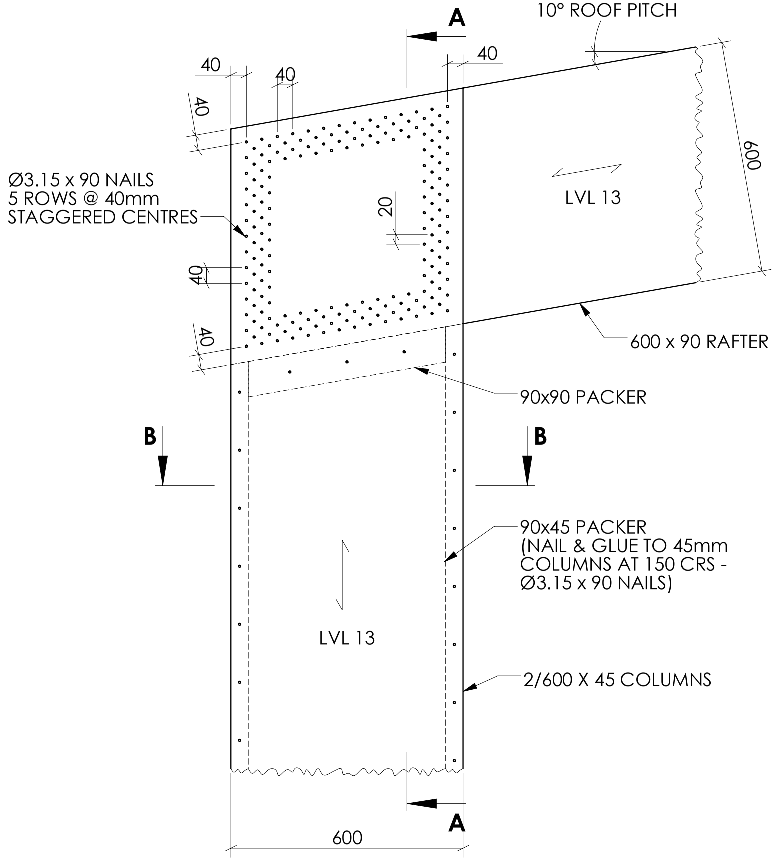
DRAWING NO.
MKCLJ - 450x90 LVL13

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	8/03/19
B	UPDATED	DJS	3/05/19
C	UPDATED	DJS	17/12/19
D	UPDATED	DJS	23/02/20

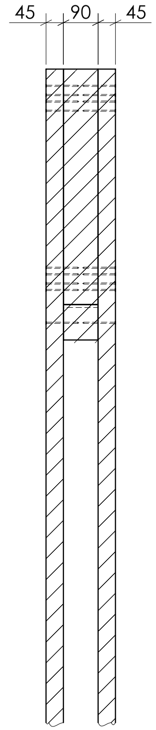
				MOMENT CAPACITY, (kNm) for K_1=1.0	
MATERIAL	RAFTER SIZE (mm)	COLUMN SIZE (mm)	ROWS OF NAILS	NAILED CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
LVL 13	600 x 90	2/600x45	4	82	161



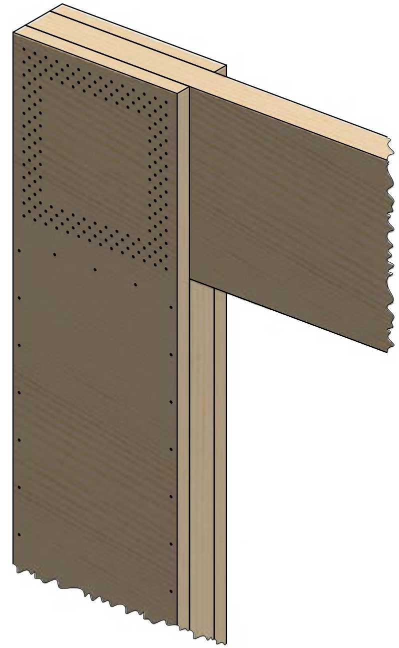
SECTION B-B



ELEVATION



SECTION A-A

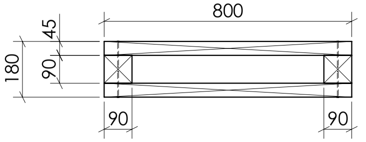


NOTES-

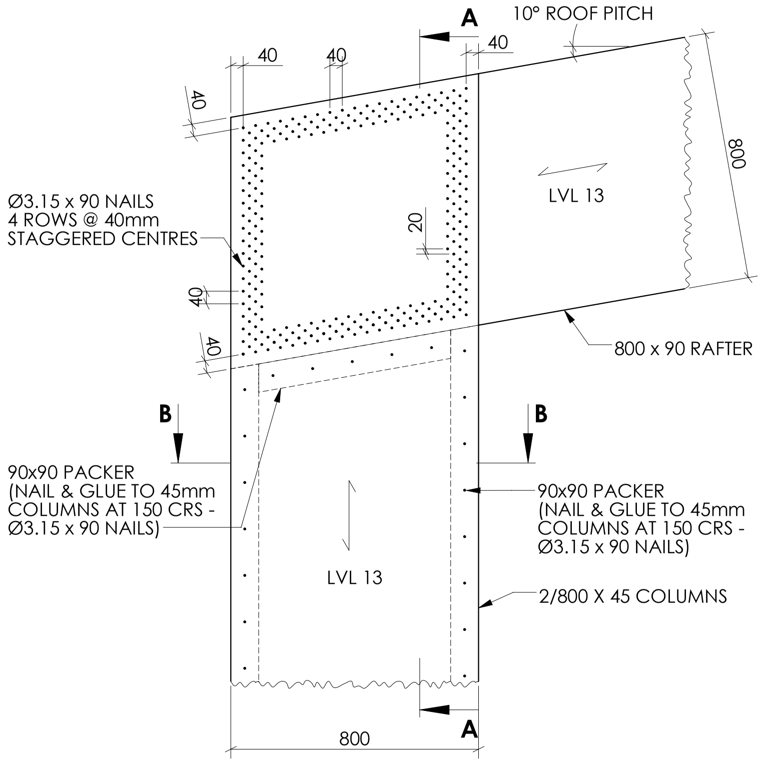
- Use flat head Ø3.15 x 90mm nails
- Head of nail to be finished flush with surface - do not over drive.
- Joint group = J5
- Ductility of connection μ=1.25-1.5 for seismic design

MOMENT KNEE CONNECTION, LAP JOINT (MKCLJ - 600x90 LVL13)	www.nzwood.co.nz design@nzwood.co.nz	DRAWING NO.	REV	REVISION DESCRIPTION	DRN	DATE
		MKCLJ - 600x90 LVL13	A	ISSUED FOR INFORMATION	DJS	8/03/19
			B	UPDATED	DJS	17/12/19
			C	UPDATED	DJS	23/02/20

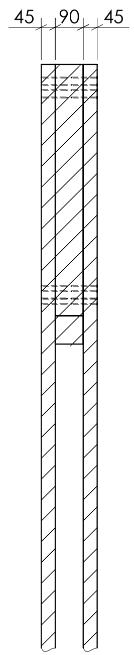
MATERIAL	RAFTER SIZE (mm)	COLUMN SIZE (mm)	ROWS OF NAILS	MOMENT CAPACITY, (kNm) for K_1=1.0 — NAILED CONNECTION CAPACITY, ØM	MOMENT CAPACITY, (kNm) for K_1=1.0 — MEMBER CAPACITY, ØM
LVL 13	800 x 90	2/800x45	4	178	272



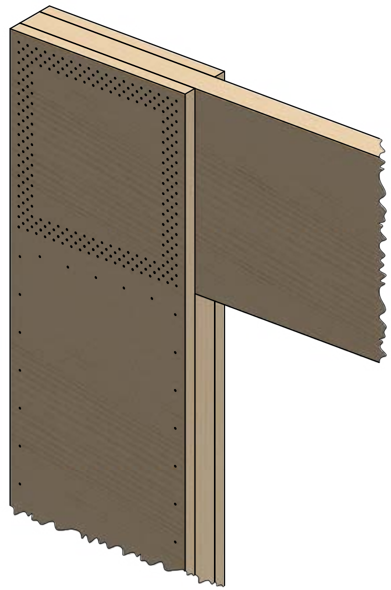
SECTION B-B



ELEVATION



SECTION A-A



NOTES-

- Use flat head Ø3.15 x 90mm nails
- Head of nail to be finished flush with surface - do not over drive.
- Joint group = J5
- Ductility of connection μ=1.25-1.5 for seismic design

MOMENT KNEE CONNECTION, LAP JOINT
(MKCLJ - 800x90 LVL13)

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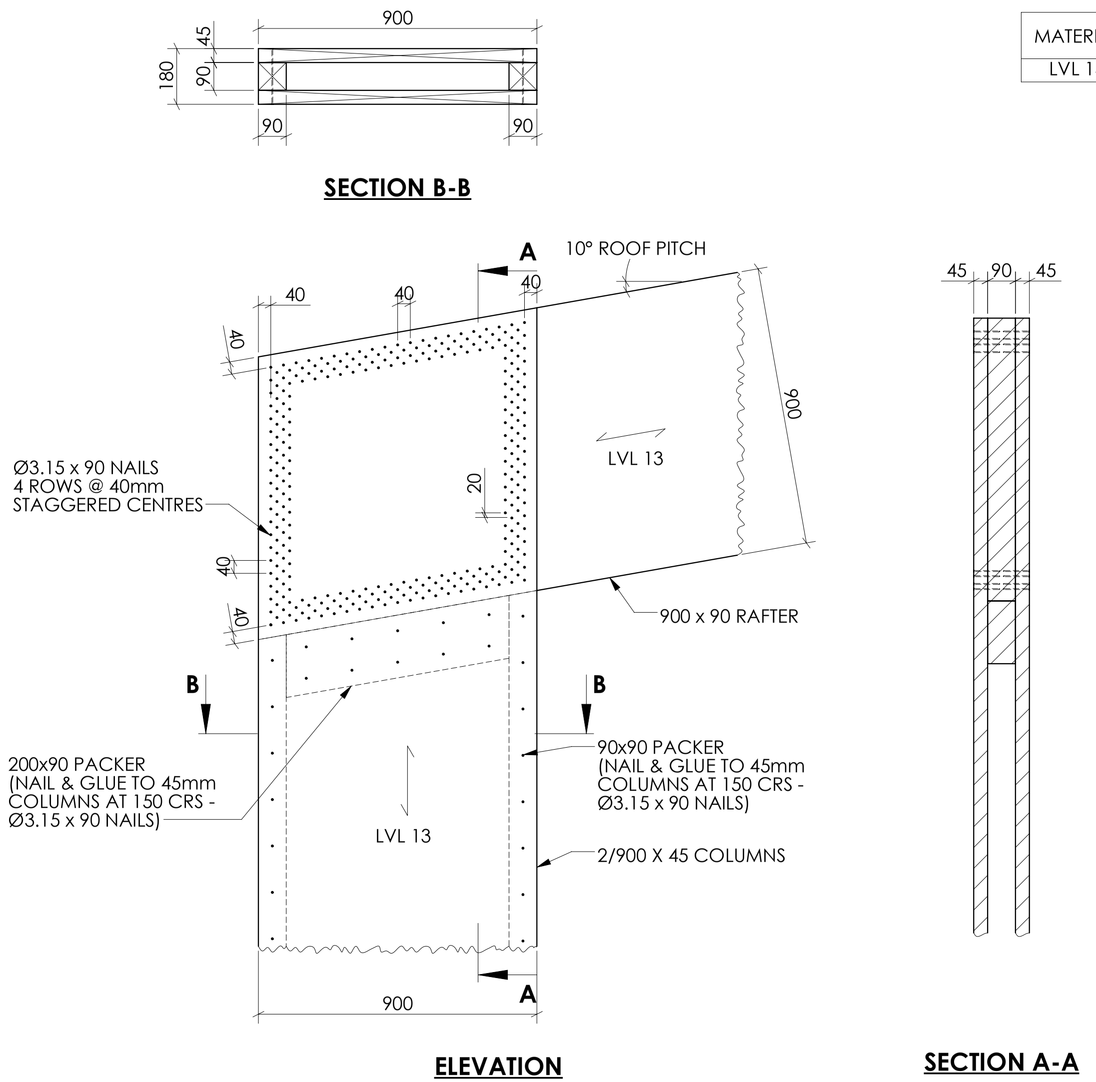
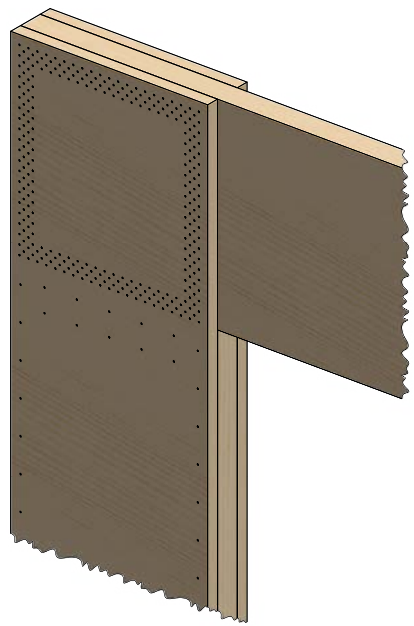
DRAWING NO.
MKCLJ - 800x90 LVL13

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	8/03/19
B	UPDATED	DJS	17/12/19
C	UPDATED	DJS	23/02/20

MATERIAL	RAFTER SIZE (mm)	COLUMN SIZE (mm)	ROWS OF NAILS	MOMENT CAPACITY, (kNm) for K_1=1.0 NAILED CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
LVL 13	900 x 90	2/900x45	4	240	338





NOTES-

- Use flat head Ø3.15 x 90mm nails
- Head of nail to be finished flush with surface - do not over drive.
- Joint group = J5
- Ductility of connection μ=1.25-1.5 for seismic design

MOMENT KNEE CONNECTION, LAP JOINT
(MKCLJ - 900x90 LVL13)

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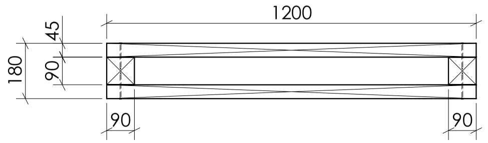
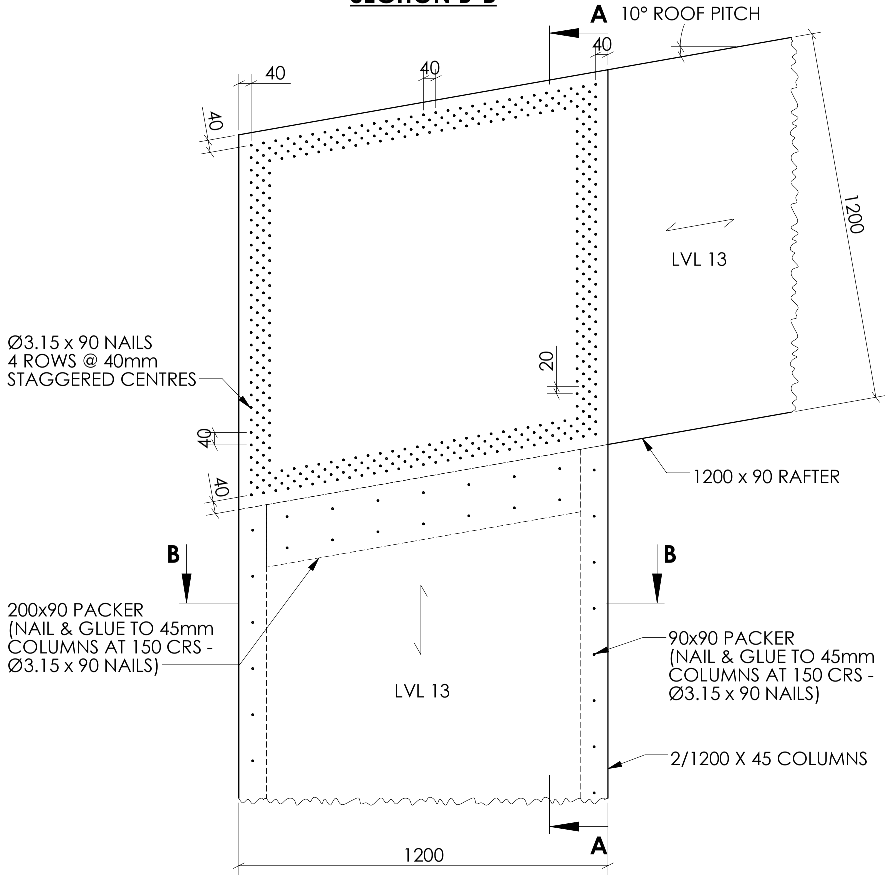
DRAWING NO.
MKCLJ - 900x90 LVL13

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	8/03/19
B	UPDATED	DJS	17/12/19
C	UPDATED	DJS	23/02/20

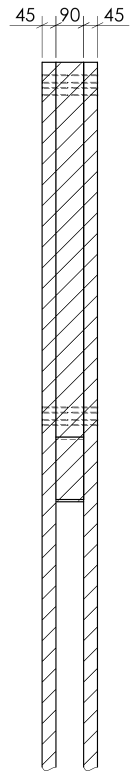
				MOMENT CAPACITY, (kNm) for K_1=1.0	
MATERIAL	RAFTER SIZE (mm)	COLUMN SIZE (mm)	ROWS OF NAILS	NAILED CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
LVL 13	1200 x 90	2/1200x45	4	479	573



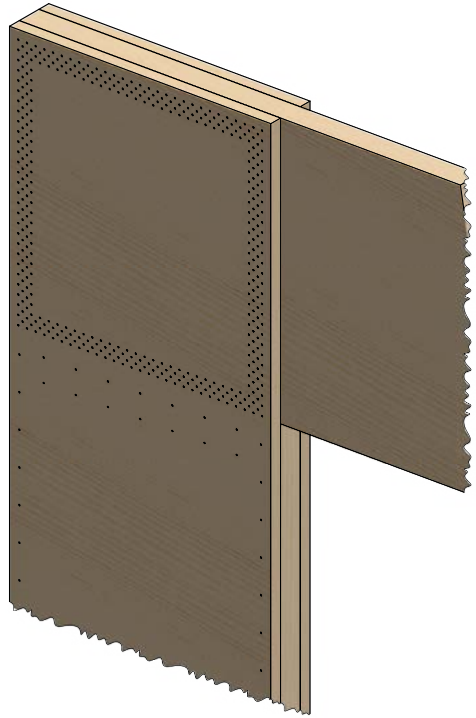
SECTION B-B



ELEVATION



SECTION A-A



NOTES-

- Use flat head Ø3.15 x 90mm nails
- Head of nail to be finished flush with surface - do not over drive.
- Joint group = J5
- Ductility of connection μ=1.25-1.5 for seismic design

MOMENT KNEE CONNECTION, LAP JOINT
(MKCLJ - 1200x90 LVL13)

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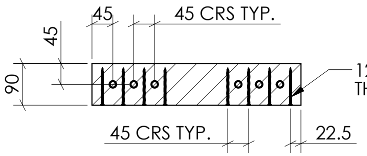
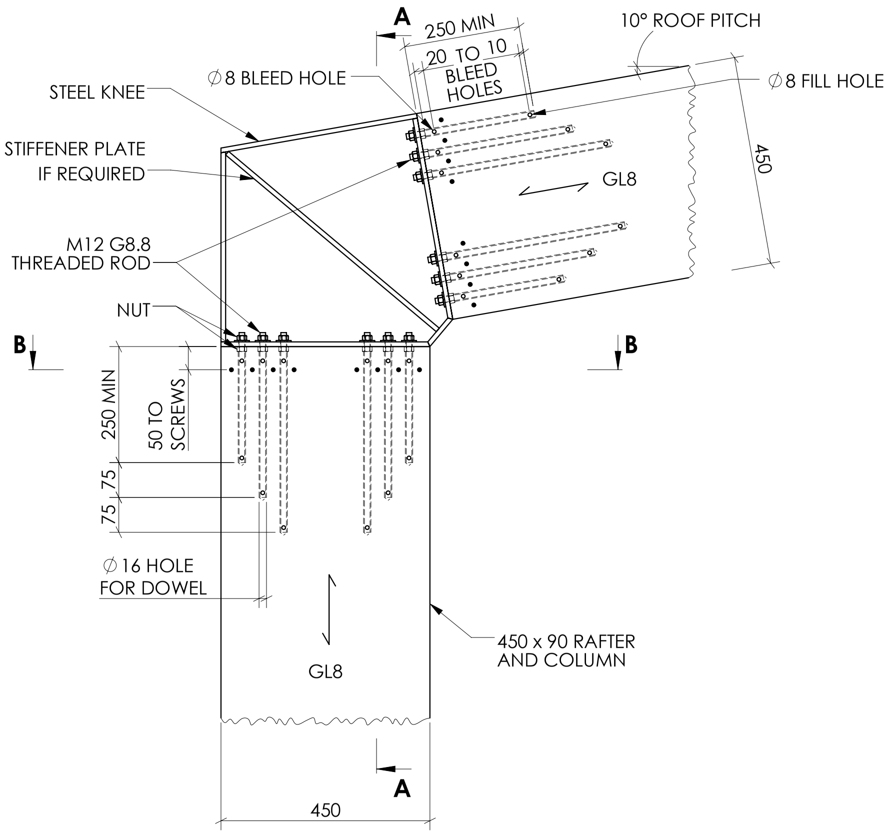
DRAWING NO.
MKCLJ - 1200x90 LVL13

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	8/03/19
B	UPDATED	DJS	17/12/19
C	UPDATED	DJS	23/02/20

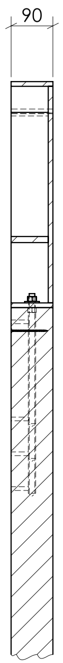
MATERIAL	MEMBER SIZE (mm)	ROD Ø(mm)	TOTAL NUMBER OF RODS PER MEMBER END	MOMENT CAPACITY, (kNm) for K_1=1.0 CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
GL8	450 x 90	12 G8.8	6	29	46



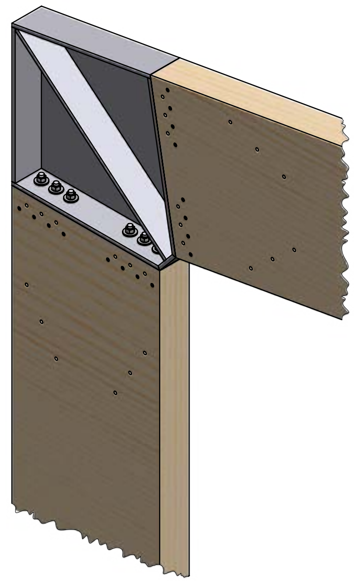
SECTION B-B



ELEVATION



SECTION A-A



NOTES-

- Epoxy rods shall be installed in factory with quality control systems in place.
- Epoxy glue options: West Systems Z105, Araldite K-80, Araldite 2005, East 221
- Rod hole Ø16mm (4mm greater than rod Ø)
- Steel knee bracket design is required.
- Use ductility of μ=1.0 for seismic design of epoxy dowels (steel knee can be designed for a higher ductilty if required. Capacity deisgn would be needed)

MOMENT KNEE CONNECTION, EPOXY ROD (MKCER - 450x90 GL8)

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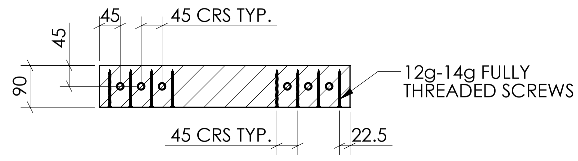
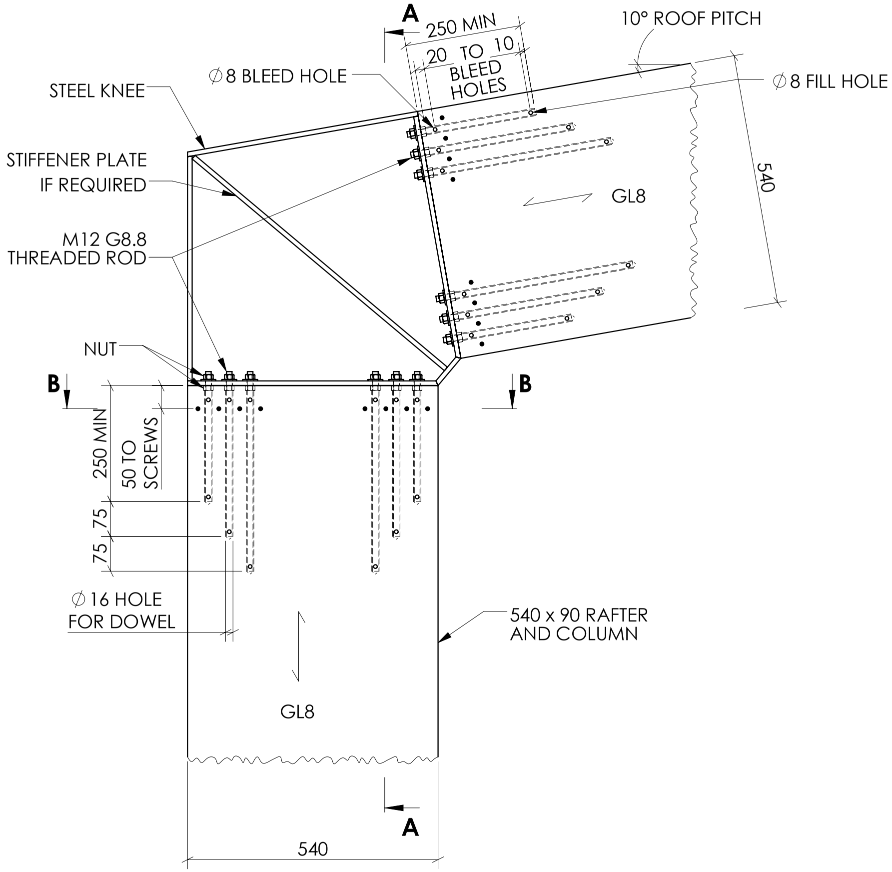
DRAWING NO. MKCER - 450x90 GL8

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	4/01/20
B	UPDATED	DJS	1/03/20
C	UPDATED	DJS	31/03/20

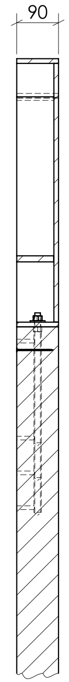


SECTION B-B

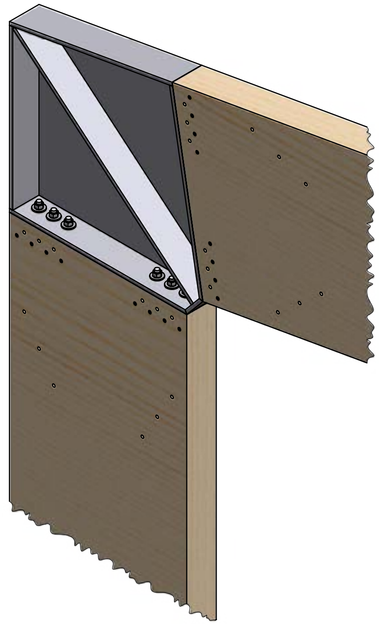
MATERIAL	MEMBER SIZE (mm)	ROD Ø(mm)	TOTAL NUMBER OF RODS PER MEMBER END	MOMENT CAPACITY, (kNm) for K_1=1.0	
				CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
GL8	540 x 90	12 G8.8	6	37	66



ELEVATION



SECTION A-A



NOTES-

- Epoxy rods shall be installed in factory with quality control systems in place.
- Epoxy glue options: West Systems Z105, Araldite K-80, Araldite 2005, East 221
- Rod hole Ø16mm (4mm greater than rod Ø)
- Steel knee bracket design is required.
- Use ductility of μ=1.0 for seismic design of epoxy dowels (steel knee can be designed for a higher ductilty if required. Capacity deisgn would be needed)

MOMENT KNEE CONNECTION, EPOXY ROD (MKCER - 540x90 GL8)

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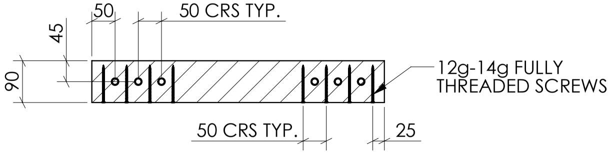
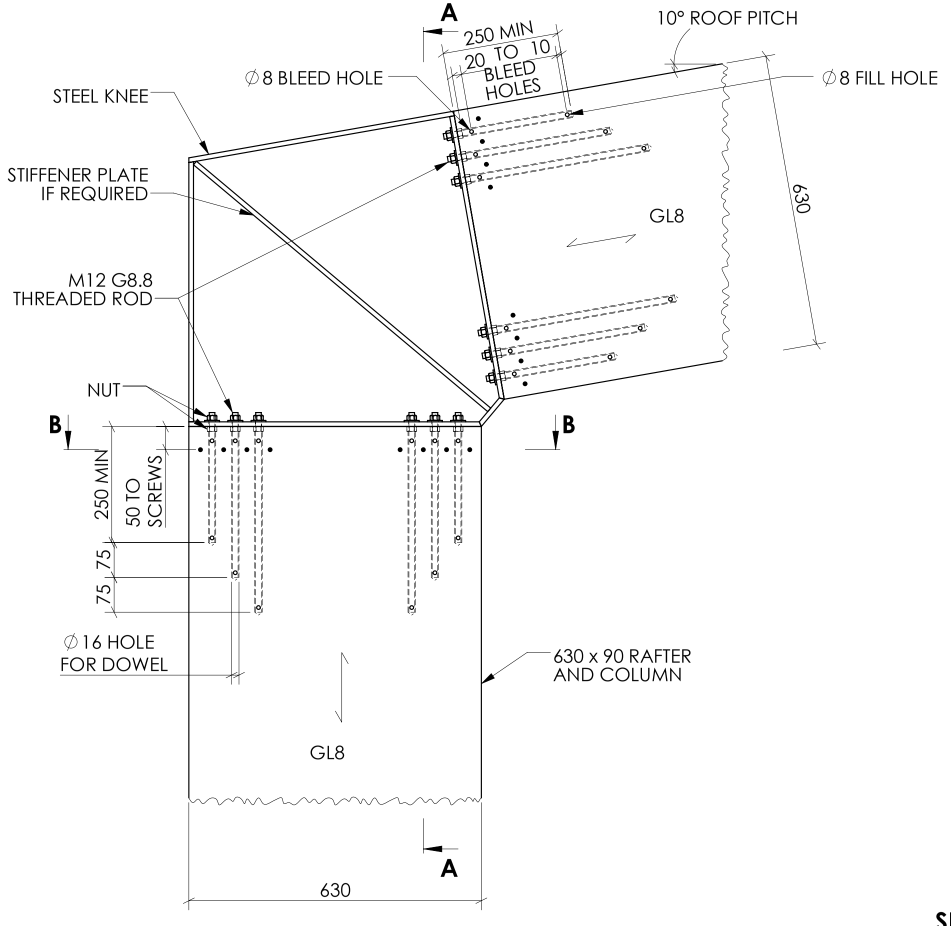
DRAWING NO.
MKCER - 540x90 GL8

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	4/01/20
B	UPDATED	DJS	1/03/20
C	UPDATED	DJS	31/03/20

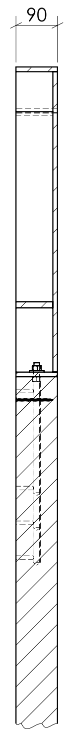


SECTION B-B

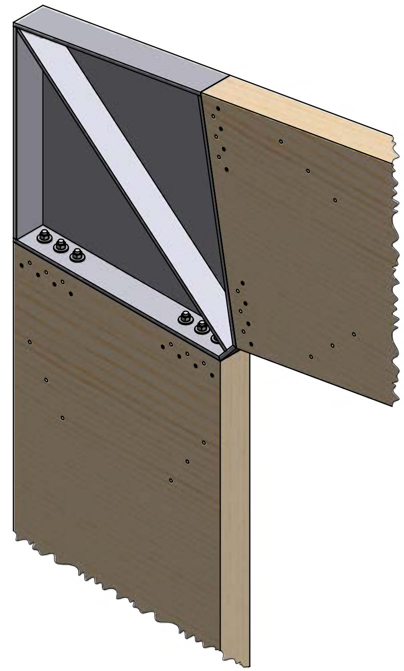
MATERIAL	MEMBER SIZE (mm)	ROD Ø(mm)	TOTAL NUMBER OF RODS PER MEMBER END	MOMENT CAPACITY, (kNm) for K_1=1.0	
				CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
GL8	630 x 90	12 G8.8	6	49	90



ELEVATION



SECTION A-A



NOTES-

- Epoxy rods shall be installed in factory with quality control systems in place.
- Epoxy glue options: West Systems Z105, Araldite K-80, Araldite 2005, East 221
- Rod hole Ø16mm (4mm greater than rod Ø)
- Steel knee bracket design is required.
- Use ductility of μ=1.0 for seismic design of epoxy dowels (steel knee can be designed for a higher ductility if required. Capacity deisgn would be needed)

MOMENT KNEE CONNECTION, EPOXY ROD (MKCER - 630x90 GL8)

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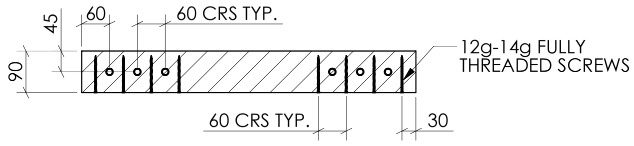
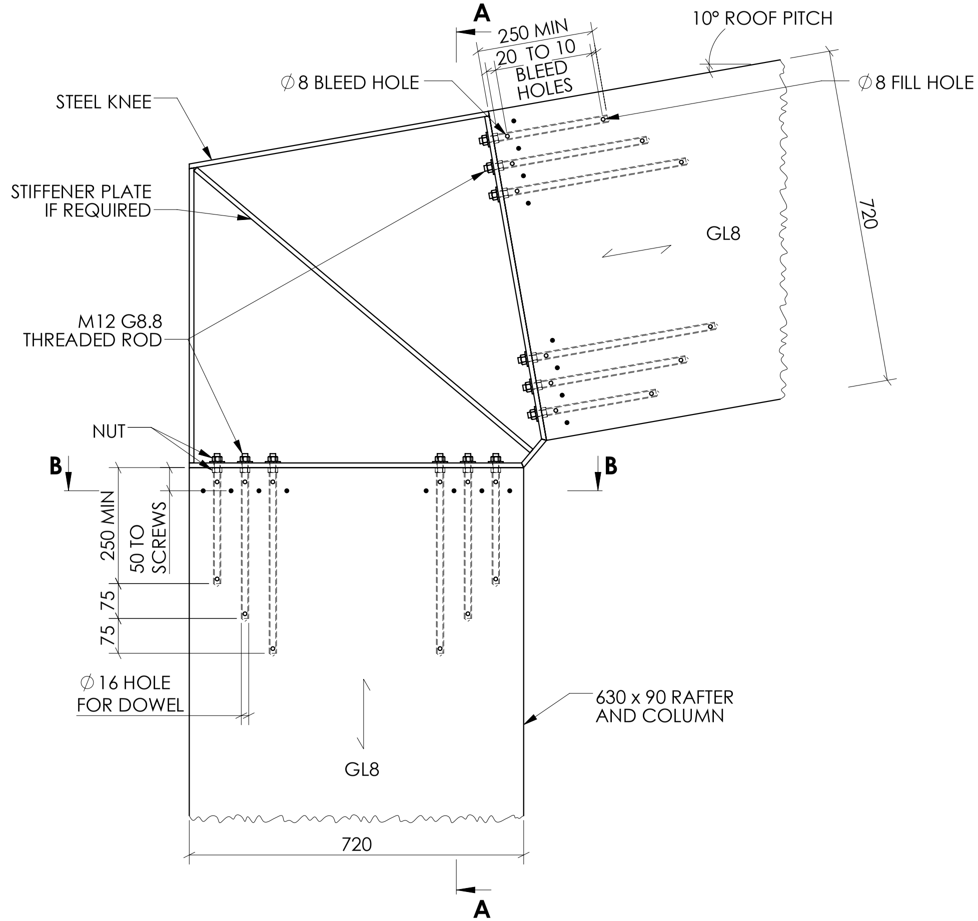
DRAWING NO. MKCER - 630x90 GL8

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	4/03/19
B	UPDATED	DJS	1/03/20
C	UPDATED	DJS	31/03/20

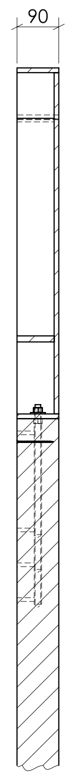


SECTION B-B

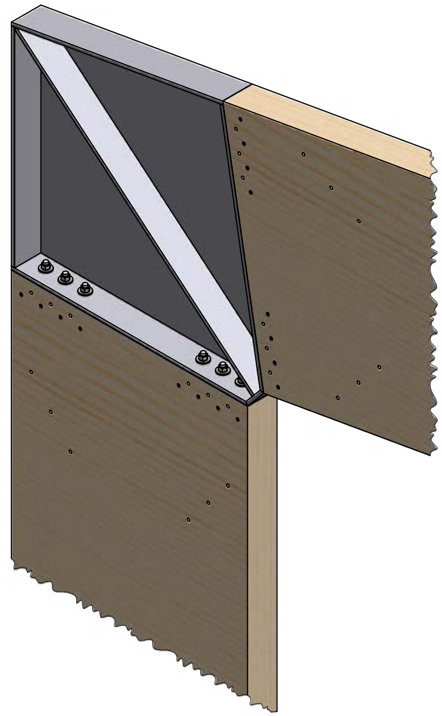
MATERIAL	MEMBER SIZE (mm)	ROD Ø(mm)	TOTAL NUMBER OF RODS PER MEMBER END	MOMENT CAPACITY, (kNm) for K_1=1.0: CONNECTION CAPACITY, ØM	MOMENT CAPACITY, (kNm) for K_1=1.0: MEMBER CAPACITY, ØM
GL8	720 x 90	12 G8.8	6	67	118



ELEVATION



SECTION A-A



NOTES-

- Epoxy rods shall be installed in factory with quality control systems in place.
- Epoxy glue options: West Systems Z105, Araldite K-80, Araldite 2005, East 221
- Rod hole Ø16mm (4mm greater than rod Ø)
- Steel knee bracket design is required.
- Use ductility of μ=1.0 for seismic design of epoxy dowels (steel knee can be designed for a higher ductilty if required. Capacity deisgn would be needed)

MOMENT KNEE CONNECTION, EPOXY ROD
(MKCER - 720x90 GL8)

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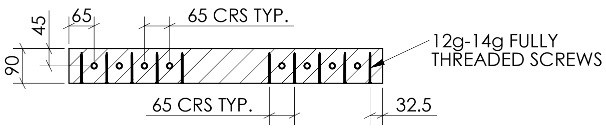
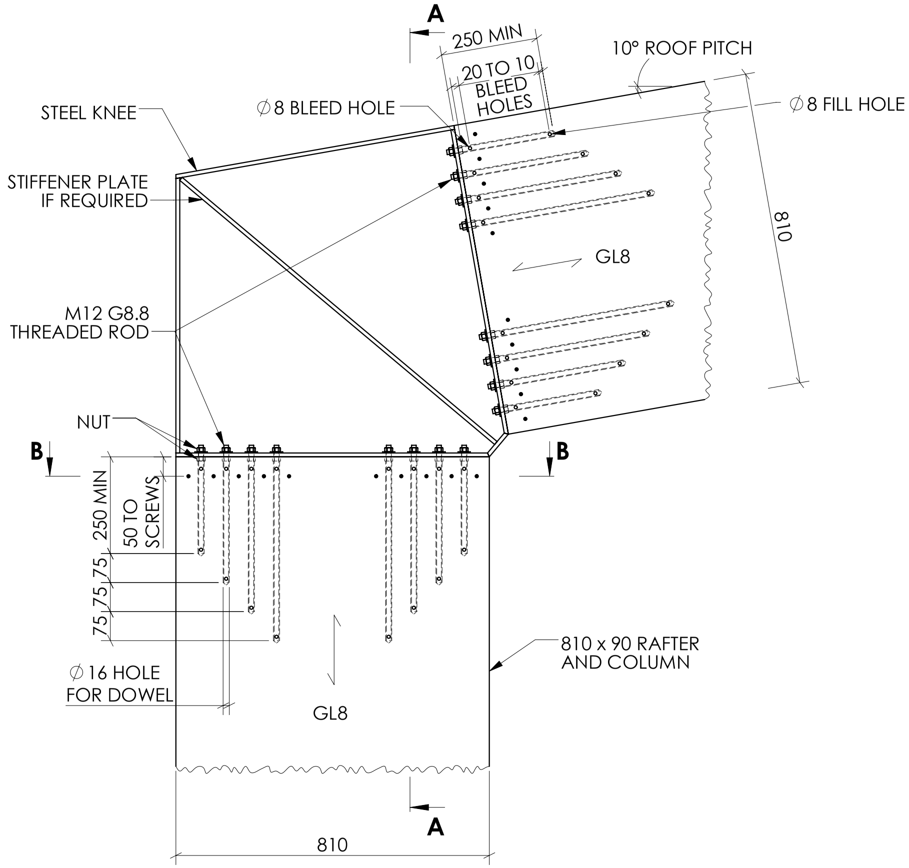
DRAWING NO.
MKCER - 720x90 GL8

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	4/03/19
B	UPDATED	DJS	1/03/20
C	UPDATED	DJS	31/03/20

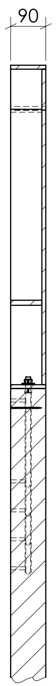


SECTION B-B

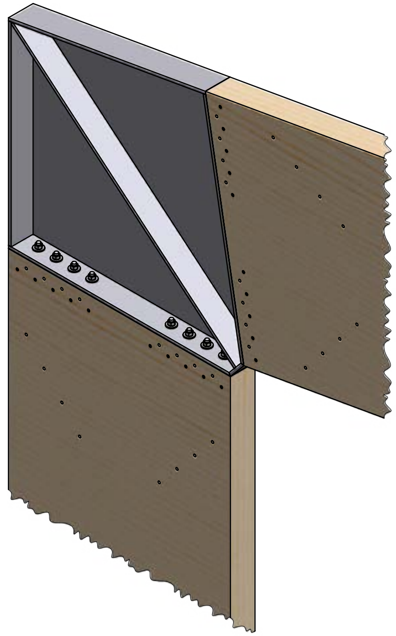
				MOMENT CAPACITY, (kNm) for K_1=1.0	
MATERIAL	MEMBER SIZE (mm)	ROD Ø(mm)	TOTAL NUMBER OF RODS PER MEMBER END	CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
GL8	810 x 90	12 G8.8	8	103	150



ELEVATION



SECTION A-A



NOTES-

- Epoxy rods shall be installed in factory with quality control systems in place.
- Epoxy glue options: West Systems Z105, Araldite K-80, Araldite 2005, East 221
- Rod hole Ø16mm (4mm greater than rod Ø)
- Steel knee bracket design is required.
- Use ductility of μ=1.0 for seismic design of epoxy dowels (steel knee can be designed for a higher ductilty if required. Capacity deisgn would be needed)

MOMENT KNEE CONNECTION, EPOXY ROD
(MKCER - 810x90 GL8)

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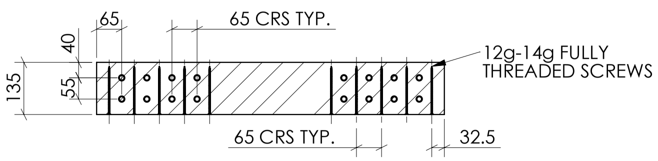
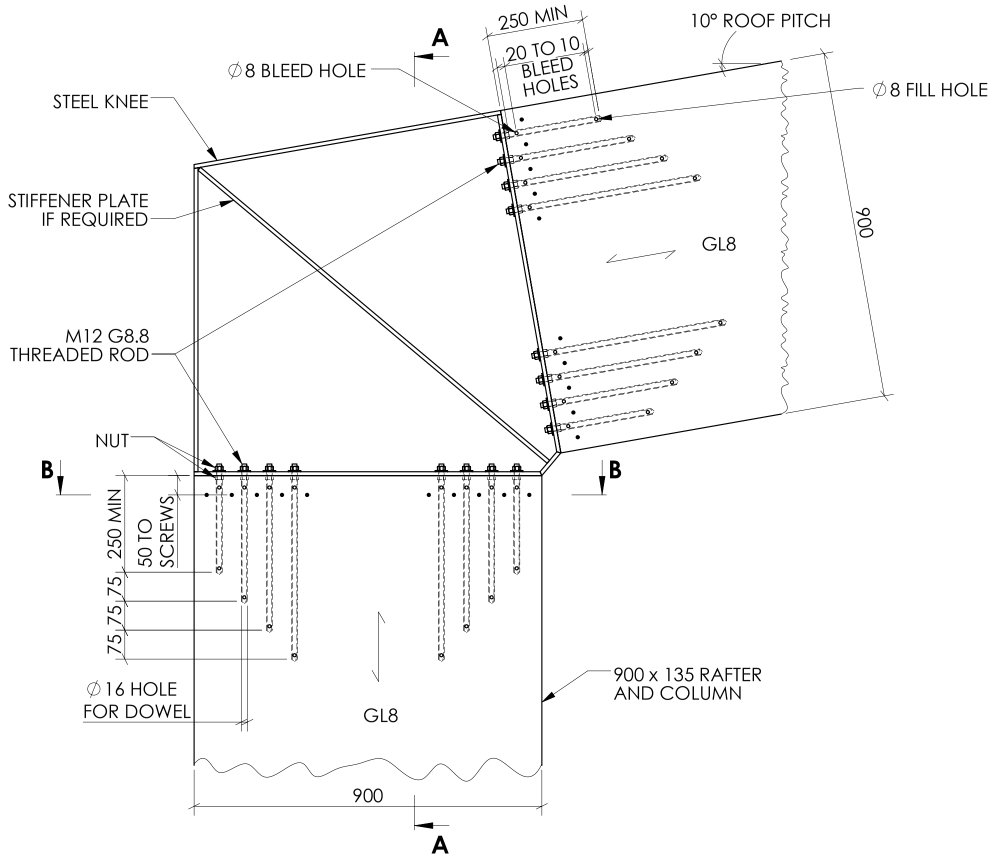
DRAWING NO.
MKCER - 810x90 GL8

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	4/01/20
B	UPDATED	DJS	2/03/20
C	UPDATED	DJS	31/03/20

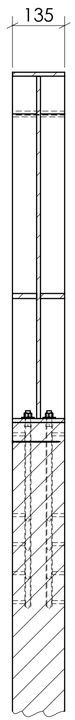


SECTION B-B

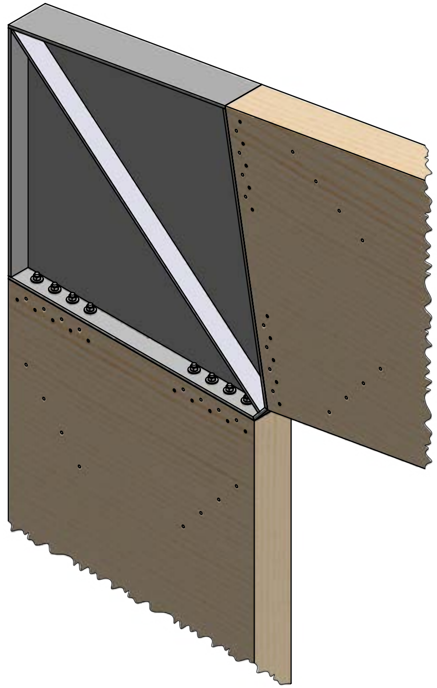
MATERIAL	MEMBER SIZE (mm)	ROD Ø(mm)	TOTAL NUMBER OF RODS PER MEMBER END	MOMENT CAPACITY, (kNm) for K_1=1.0	
				CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
GL8	900 x 135	12 G8.8	8	178	277



ELEVATION



SECTION A-A



NOTES-

- Epoxy rods shall be installed in factory with quality control systems in place.
- Epoxy glue options: West Systems Z105, Araldite K-80, Araldite 2005, East 221
- Rod hole Ø16mm (4mm greater than rod Ø)
- Steel knee bracket design is required.
- Use ductility of μ=1.0 for seismic design of epoxy dowels (steel knee can be designed for a higher ductilty if required. Capacity deisgn would be needed)

MOMENT KNEE CONNECTION, EPOXY ROD
(MKCER - 900x135 GL8)

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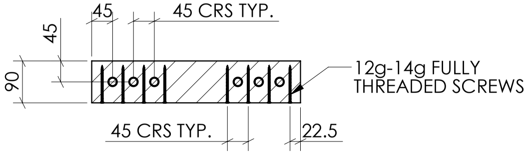
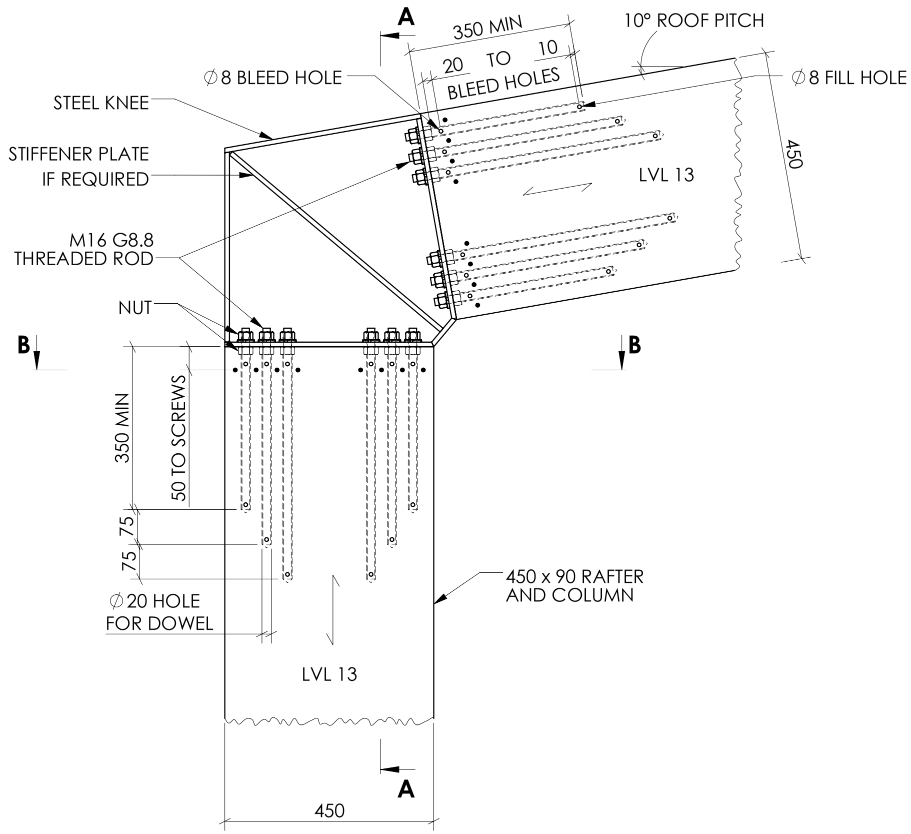
DRAWING NO.
MKCER - 900x135 GL8

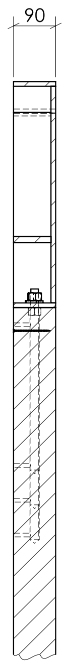
REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	4/01/20
B	UPDATED	DJS	2/03/20
C	UPDATED	DJS	31/03/20



SECTION B-B

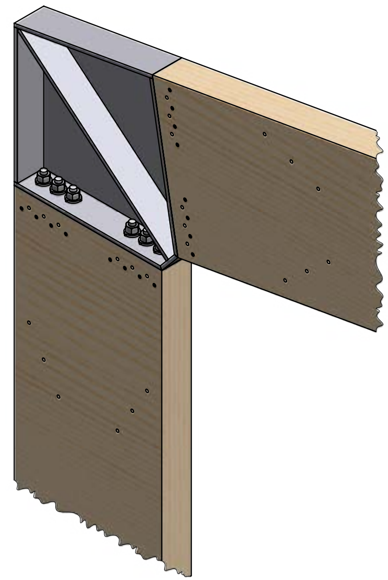


ELEVATION



SECTION A-A

MATERIAL	MEMBER SIZE (mm)	ROD Ø(mm)	TOTAL NUMBER OF RODS PER MEMBER END	MOMENT CAPACITY, (kNm) for K_1=1.0 CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
LVL 13	450 x 90	16 G8.8	6	72	95



NOTES-

- Epoxy rods shall be installed in factory with quality control systems in place.
- Epoxy glue options: West Systems Z105, Araldite K-80, Araldite 2005, East 221
- Rod hole Ø20mm (4mm greater than rod Ø)
- Steel knee bracket design is required.
- Use ductility of μ=1.0 for seismic design of epoxy dowels (steel knee can be designed for a higher ductility if required. Capacity deisgn would be needed)

MOMENT KNEE CONNECTION, EPOXY ROD (MKCER - 450x90 LVL13)

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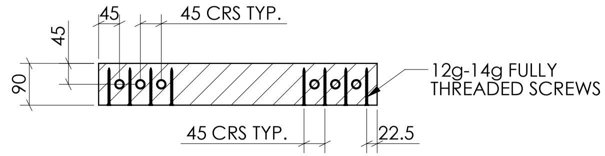
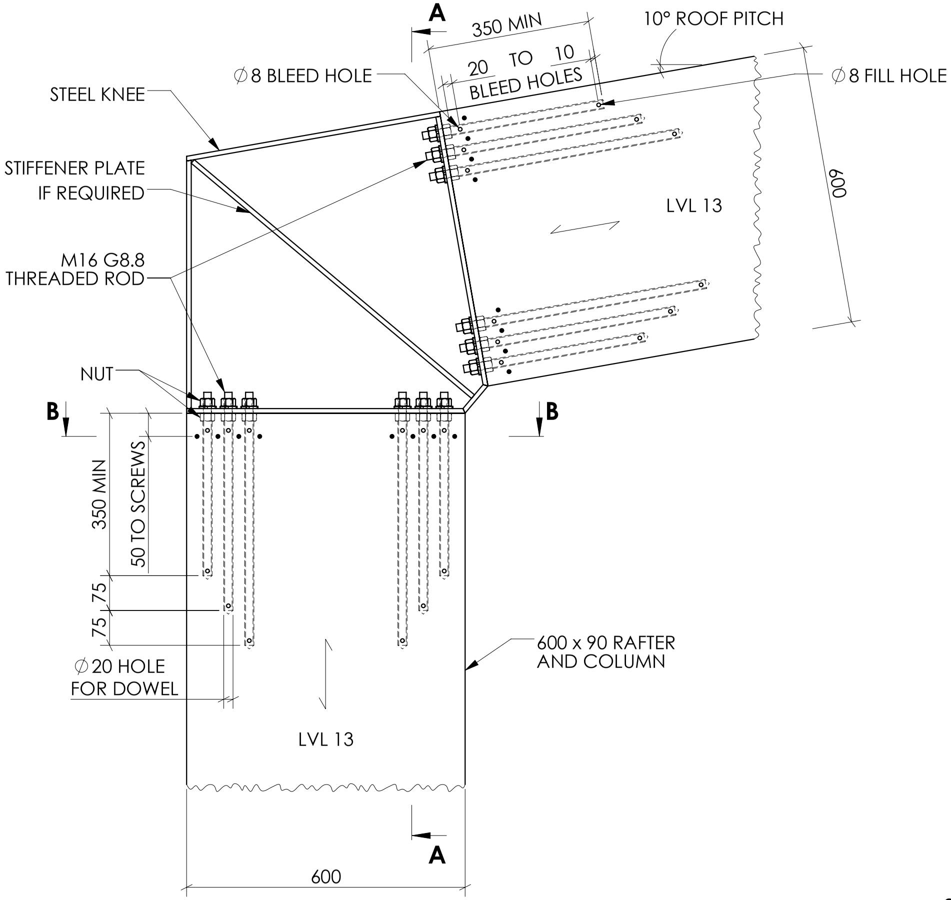
DRAWING NO.: MKCER - 450x90 LVL13

REV	REVISION DESCRIPTION	DRN	DATE
B	UPDATED	DJS	3/05/19
C	UPDATED	DJS	4/01/20
D	UPDATED	DJS	23/02/20
E	UPDATED	DJS	31/03/20

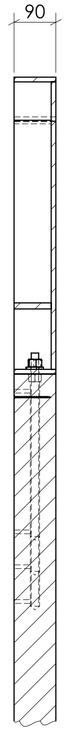


SECTION B-B

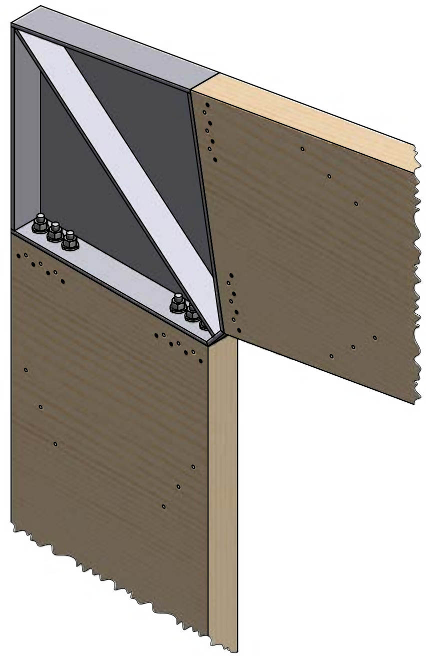
				MOMENT CAPACITY, (kNm) for K_1=1.0	
MATERIAL	MEMBER SIZE (mm)	ROD Ø(mm)	TOTAL NUMBER OF RODS PER MEMBER END	CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
LVL 13	600 x 90	16 G8.8	6	104	161



ELEVATION



SECTION A-A



NOTES-

- Epoxy rods shall be installed in factory with quality control systems in place.
- Epoxy glue options: West Systems Z105, Araldite K-80, Araldite 2005, East 221
- Rod hole Ø20mm (4mm greater than rod Ø)
- Steel knee bracket design is required.
- Use ductility of μ=1.0 for seismic design of epoxy dowels (steel knee can be designed for a higher ductility if required. Capacity deisgn would be needed)

MOMENT KNEE CONNECTION, EPOXY ROD
(MKCER - 600x90 LVL13)

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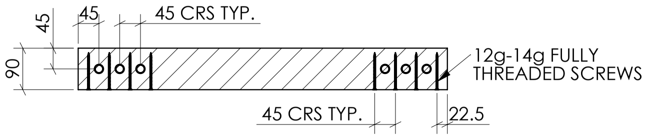
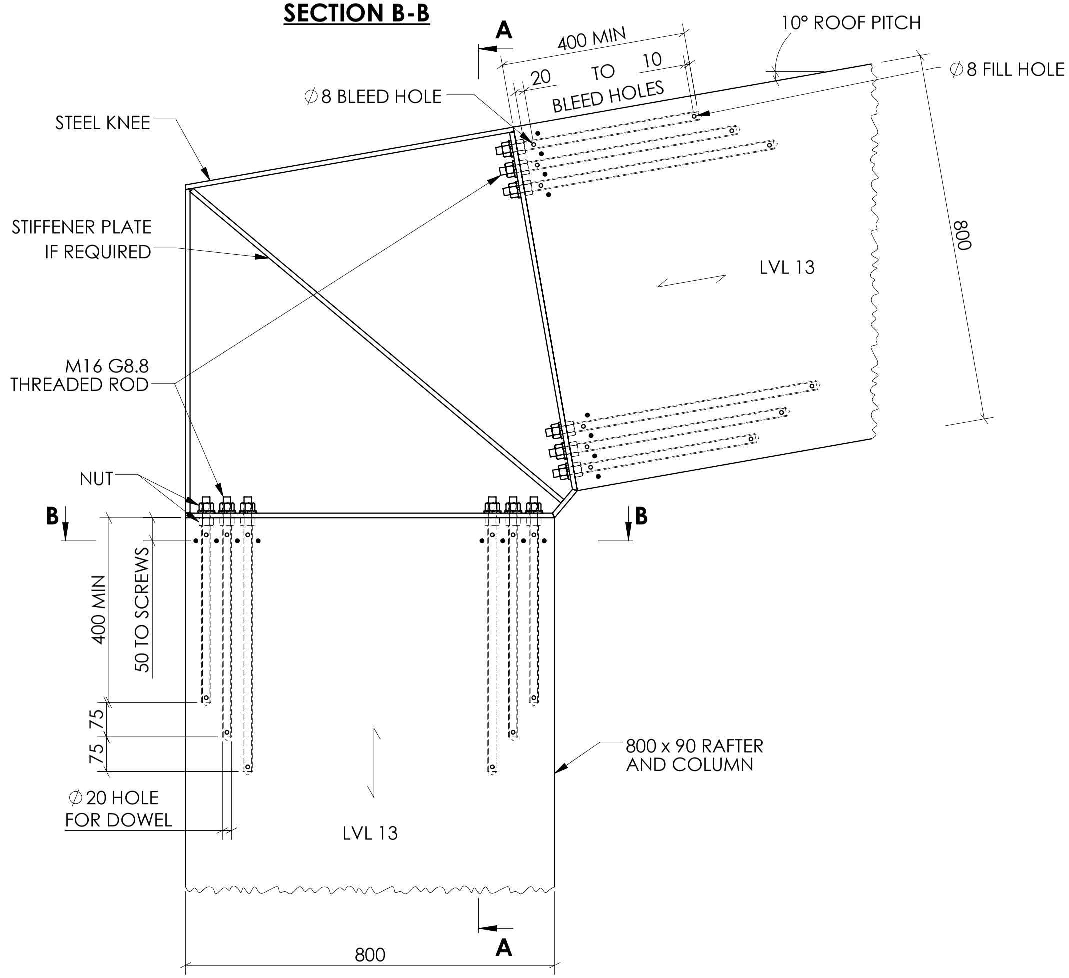
DRAWING NO.
MKCER - 600x90 LVL13

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	4/03/19
B	UPDATED	DJS	4/01/20
C	UPDATED	DJS	23/02/20
D	UPDATED	DJS	31/03/20

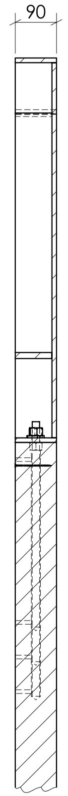


SECTION B-B

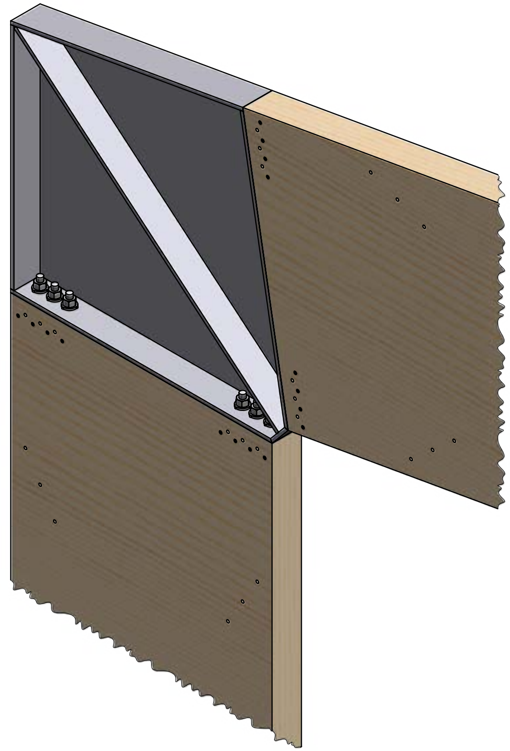
MATERIAL	MEMBER SIZE (mm)	ROD Ø(mm)	TOTAL NUMBER OF RODS PER MEMBER END	MOMENT CAPACITY, (kNm) for K_1=1.0	
				CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
LVL 13	800 x 90	16 G8.8	6	148	272



ELEVATION



SECTION A-A



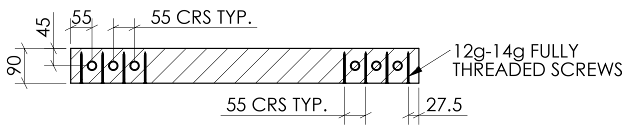
NOTES-

- Epoxy rods shall be installed in factory with quality control systems in place.
- Epoxy glue options: West Systems Z105, Araldite K-80, Araldite 2005, East 221
- Rod hole Ø20mm (4mm greater than rod Ø)
- Steel knee bracket design is required.
- Use ductility of μ=1.0 for seismic design of epoxy dowels (steel knee can be designed for a higher ductilty if required. Capacity deisgn would be needed)

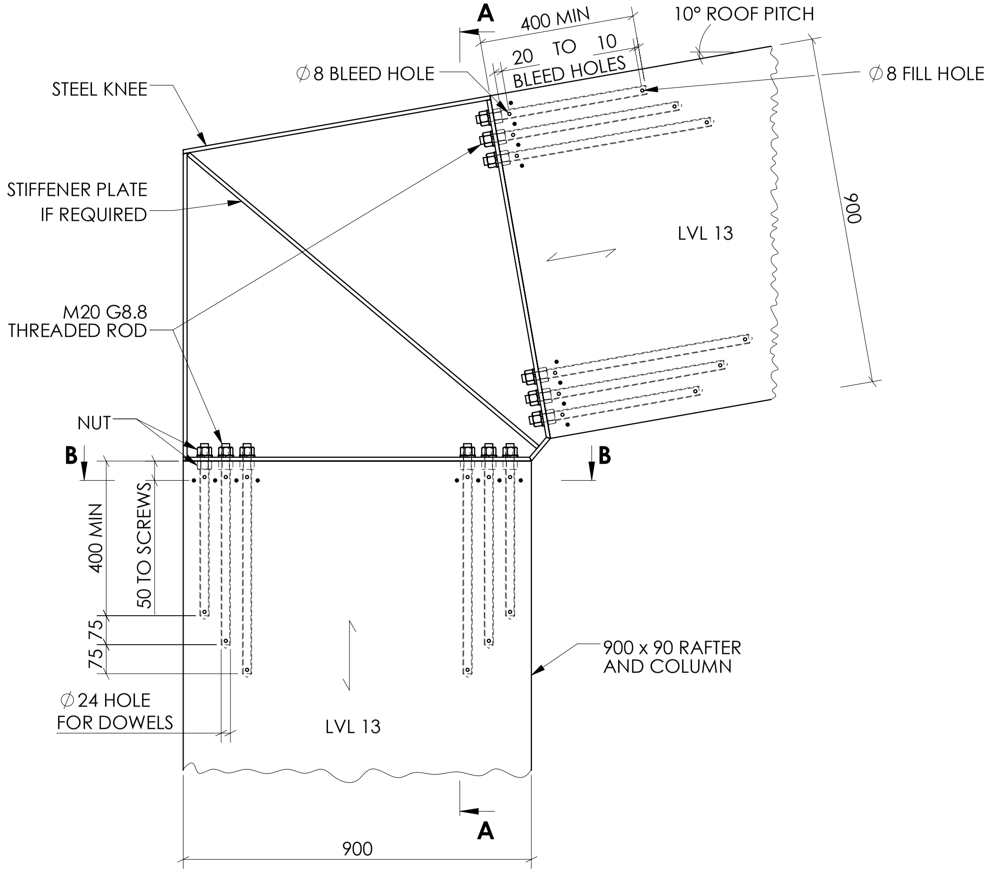
MOMENT KNEE CONNECTION, EPOXY ROD (MKCER - 800x90 LVL13)	NZ WOOD For a better world	www.nzwood.co.nz design@nzwood.co.nz	DRAWING NO. MKCER - 800x90 LVL13

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	4/03/19
B	UPDATED	DJS	4/01/20
C	UPDATED	DJS	23/02/20
D	UPDATED	DJS	31/03/20

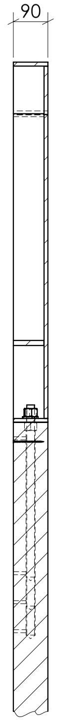
MATERIAL	MEMBER SIZE (mm)	ROD Ø(mm)	TOTAL NUMBER OF RODS PER MEMBER END	MOMENT CAPACITY, (kNm) for K_1=1.0 CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
LVL 13	900 x 90	20 G8.8	6	198	338



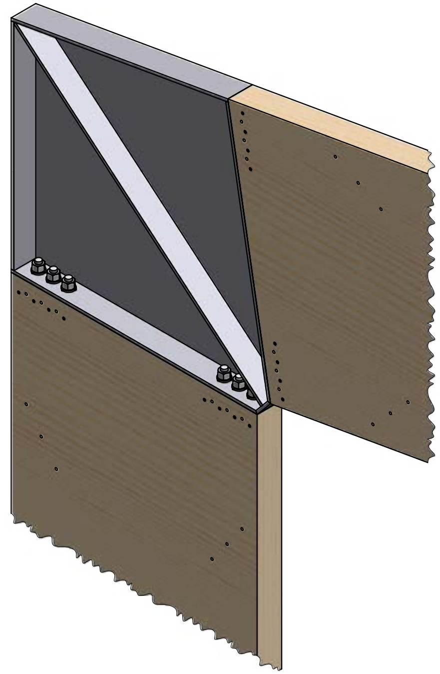
SECTION B-B



ELEVATION



SECTION A-A



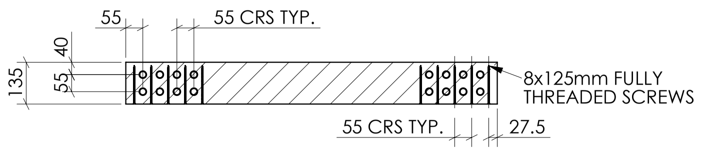
NOTES-

- Epoxy rods shall be installed in factory with quality control systems in place.
- Epoxy glue options: West Systems Z105, Araldite K-80, Araldite 2005, East 221
- Rod hole Ø24mm (4mm greater than rod Ø)
- Steel knee bracket design is required.
- Use ductility of μ=1.0 for seismic design of epoxy dowels (steel knee can be designed for a higher ductility if required. Capacity deisgn would be needed)

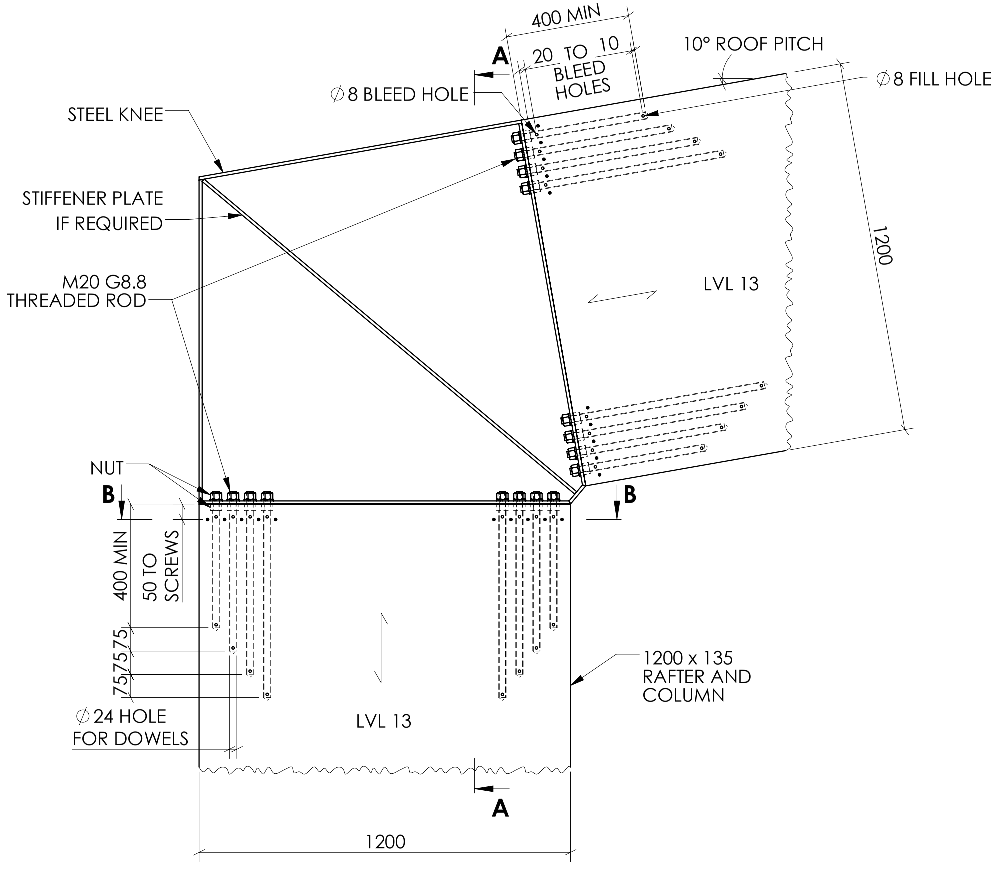
MOMENT KNEE CONNECTION, EPOXY ROD (MKCER - 900x90 LVL13)	NZ Wood For a better world www.nzwood.co.nz design@nzwood.co.nz	DRAWING NO. MKCER - 900x90 LVL13

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	4/03/19
B	UPDATED	DJS	4/01/20
C	UPDATED	DJS	23/02/20
D	UPDATED	DJS	31/03/20

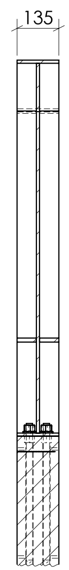
MATERIAL	MEMBER SIZE (mm)	ROD Ø(mm)	TOTAL NUMBER OF RODS PER MEMBER END	MOMENT CAPACITY, (kNm) for K_1=1.0	
				CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
LVL 13	1200 x 135	20 G8.8	16	511	859



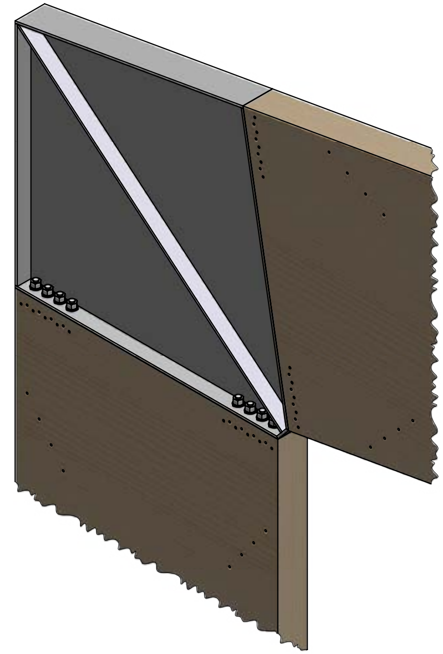
SECTION B-B



ELEVATION



SECTION A-A



NOTES-

- Epoxy rods shall be installed in factory with quality control systems in place.
- Epoxy glue options: West Systems Z105, Araldite K-80, Araldite 2005, East 221
- Rod hole Ø24mm (4mm greater than rod Ø)
- Steel knee bracket design is required.
- Use ductility of μ=1.0 for seismic design of epoxy dowels (steel knee can be designed for a higher ductilty if required. Capacity deisgn would be needed)

MOMENT KNEE CONNECTION, EPOXY ROD
(MKCER - 1200x135 LVL13)

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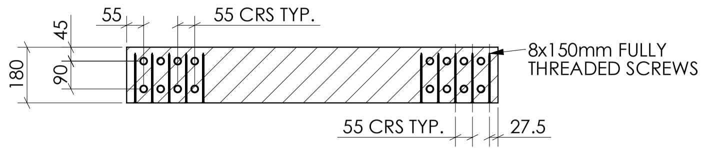
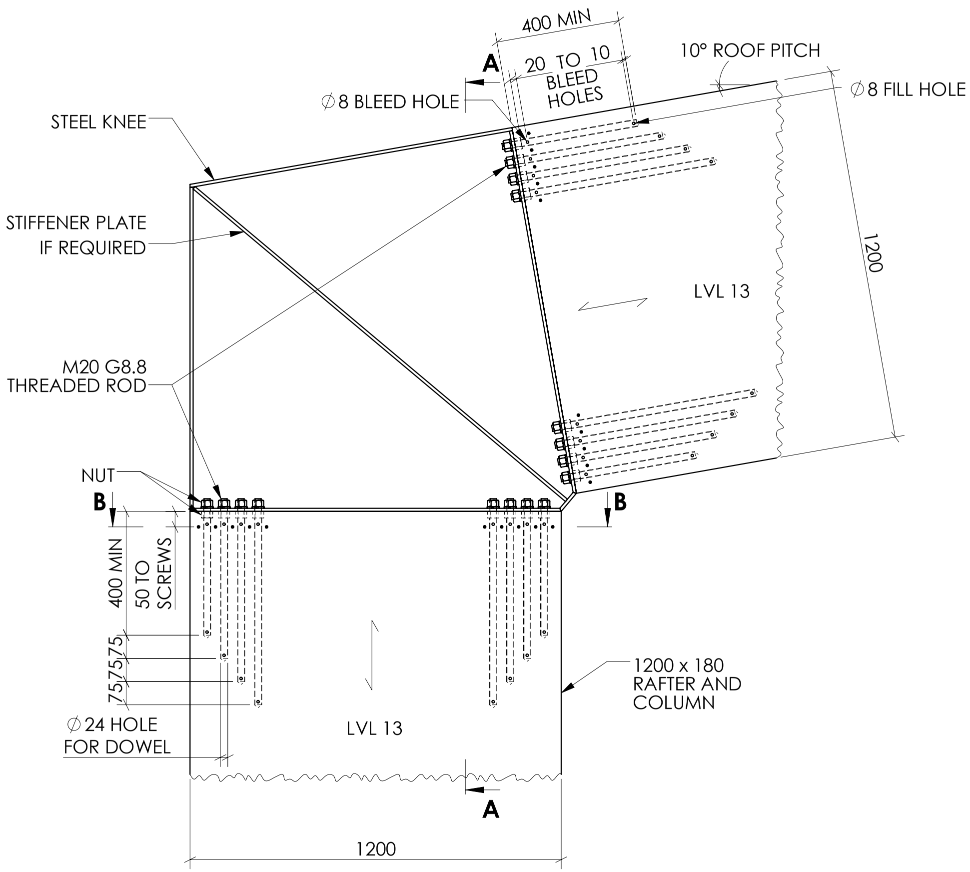
DRAWING NO.	REV	REVISION DESCRIPTION	DRN	DATE
MKCER - 1200x135 LVL13	A	ISSUED FOR INFORMATION	DJS	4/03/19
	B	UPDATED	DJS	4/01/20
	C	UPDATED	DJS	23/02/20
	D	UPDATED	DJS	31/03/20

MATERIAL	MEMBER SIZE (mm)	ROD Ø(mm)	TOTAL NUMBER OF RODS PER MEMBER END	MOMENT CAPACITY, (kNm) for K_1=1.0 CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
LVL 13	1200 x 180	20 G8.8	16	655	1146



SECTION B-B



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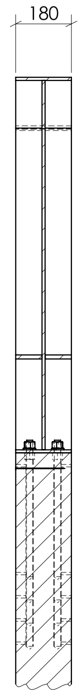
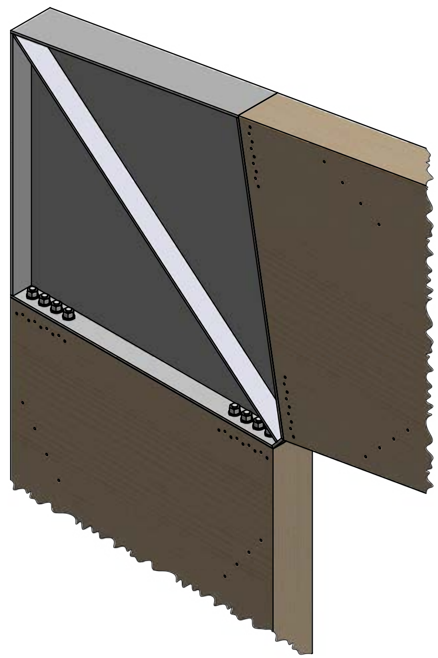


SECTION A-A



NOTES-

- Epoxy rods shall be installed in factory with quality control systems in place.
- Epoxy glue options: West Systems Z105, Araldite K-80, Araldite 2005, East 221
- Rod hole Ø24mm (4mm greater than rod Ø)
- Steel knee bracket design is required.
- Use ductility of μ=1.0 for seismic design of epoxy dowels (steel knee can be designed for a higher ductility if required. Capacity deisgn would be needed)

MOMENT KNEE CONNECTION, EPOXY ROD
(MKCER - 1200x180 LVL13)

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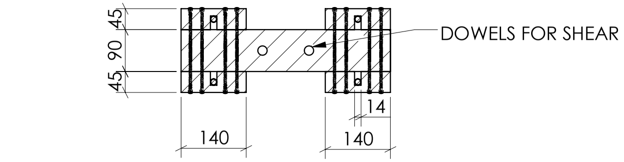
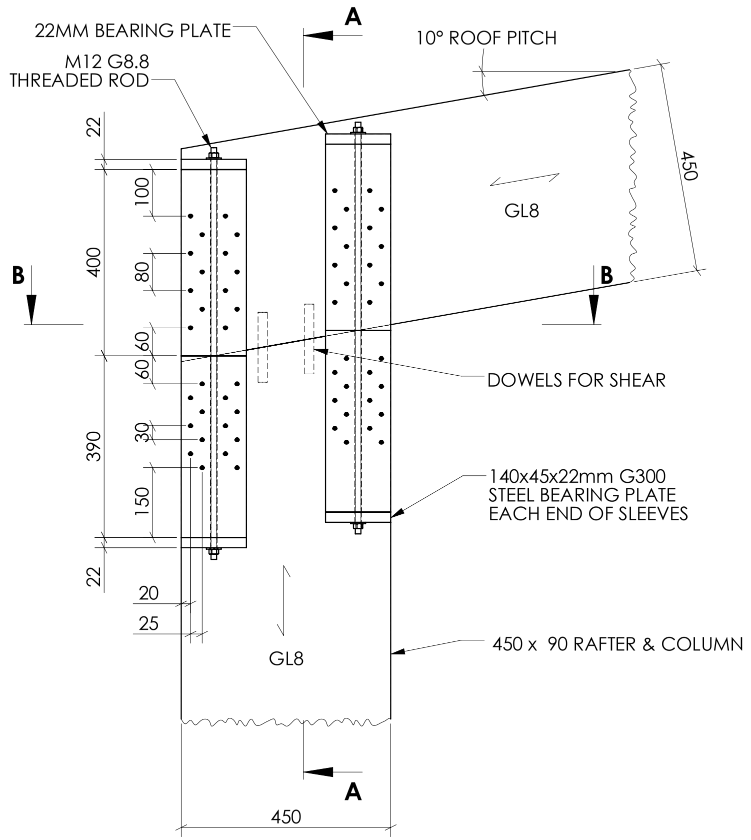
DRAWING NO.	REV	REVISION DESCRIPTION	DRN	DATE
MKCER - 1200x180 LVL13	A	ISSUED FOR INFORMATION	DJS	4/03/19
	B	UPDATED	DJS	4/01/20
	C	UPDATED	DJS	23/02/20
	D	UPDATED	DJS	31/03/20

MATERIAL	MEMBER SIZE (mm)	SCREWS FULLY THREADED IN BEAM	SCREWS FULLY THREADED IN COLUMN	MOMENT CAPACITY, (kNm) for K_1=1.0	
				CONNECTION CAPACITY,ØM	MEMBER CAPACITY, ØM
GL8	450 x 90	14/Ø6x140/SLEEVE	14/Ø6x140/SLEEVE	30	46



SECTION B-B



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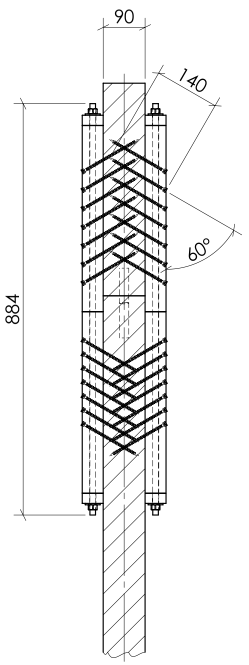
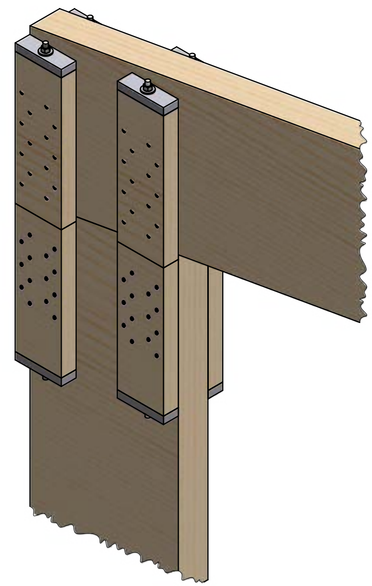


SECTION A-A



NOTES-

- Screws to be fully threaded options Spax, Rothoblaas, Wurth, etc)
- Rods G8.8 M12
- Design required for dowels to carry shear
- Connections can relax once loaded, tightening of rods following loading can reduce connection rotation
- Ductility of connection μ=1.0 for seismic design

MOMENT KNEE CONNECTION, QUICK CONNECT
(MKCQC - 450x90 GL8)

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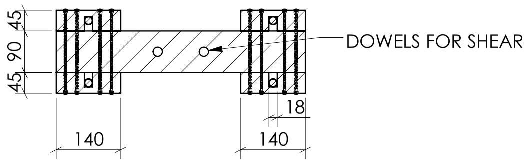
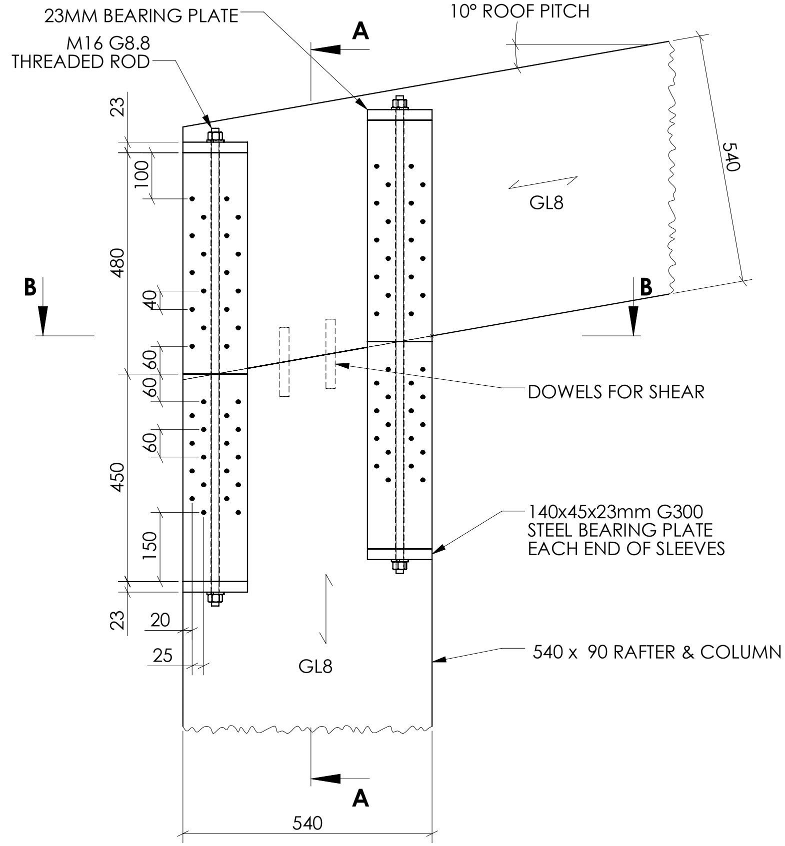
DRAWING NO.	REV	REVISION DESCRIPTION	DRN	DATE
MKCQC - 450x90 GL8	A	ISSUED FOR INFORMATION	DJS	5/01/20
	B	UPDATED	DJS	5/03/20
	C	UPDATED	DJS	1/04/20

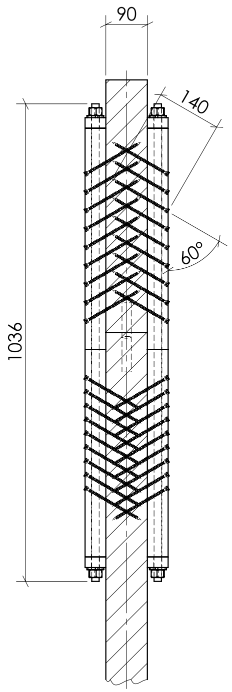
MATERIAL	MEMBER SIZE (mm)	SCREWS FULLY THREADED IN BEAM	SCREWS FULLY THREADED IN COLUMN	MOMENT CAPACITY, (kNm) for K_1=1.0 CONNECTION CAPACITY,ØM	MOMENT CAPACITY, (kNm) for K_1=1.0 MEMBER CAPACITY, ØM
GL8	540 x 90	18/Ø6x140/SLEEVE	18/Ø6x140/SLEEVE	50	65



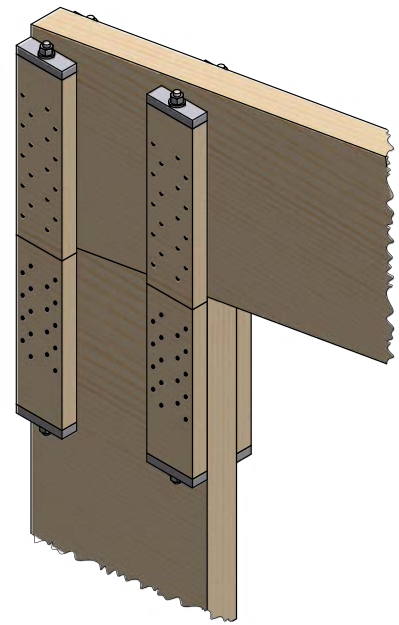
SECTION B-B



ELEVATION



SECTION A-A



NOTES-

- Screws to be fully threaded options Spax, Rothoblaas, Wurth, etc)
- Rods G8.8 M12
- Design required for dowels to carry shear
- Connections can relax once loaded, tightening of rods following loading can reduce connection rotation
- Ductility of connection μ=1.0 for seismic design

MOMENT KNEE CONNECTION, QUICK CONNECT
(MKCQC - 540x90 GL8)

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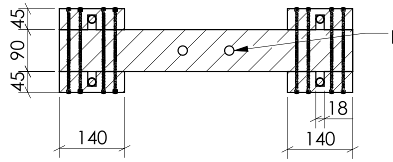
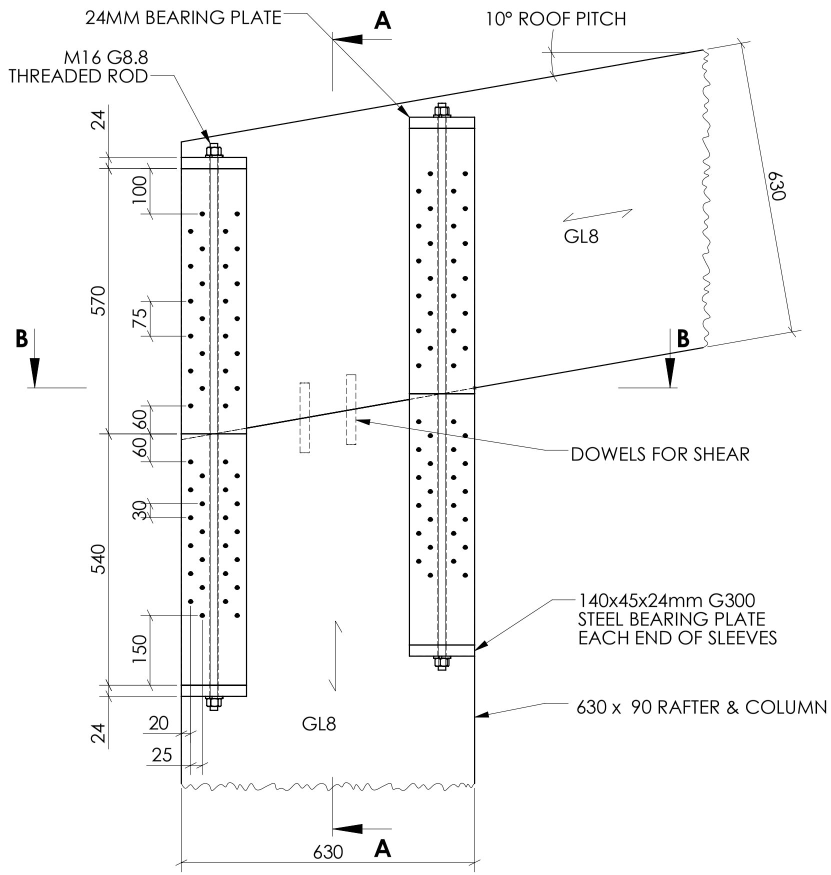
DRAWING NO.
MKCQC - 540x90 GL8

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	5/01/20
B	UPDATED	DJS	5/03/20
C	UPDATED	DJS	1/04/20

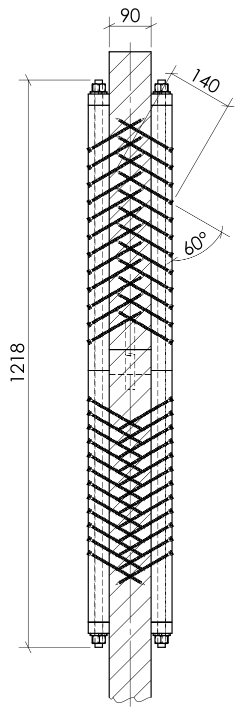
				MOMENT CAPACITY, (kNm) for K_1=1.0	
MATERIAL	MEMBER SIZE (mm)	SCREWS FULLY THREADED IN BEAM	SCREWS FULLY THREADED IN COLUMN	CONNECTION CAPACITY,ØM	MEMBER CAPACITY, ØM
GL8	630 x 90	24/Ø6x140/SLEEVE	24/Ø6x140/SLEEVE	85	90



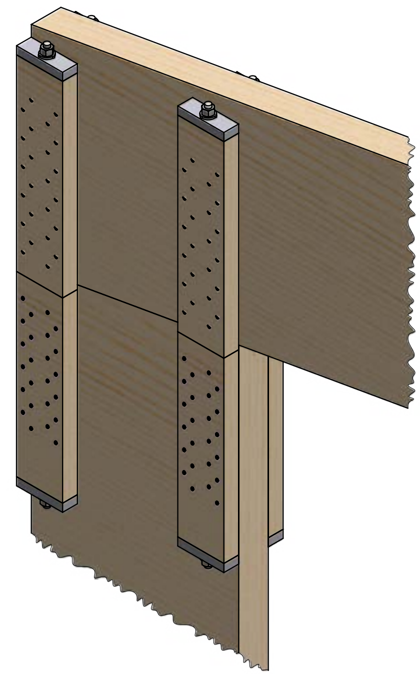
SECTION B-B



ELEVATION



SECTION A-A



NOTES-

- Screws to be fully threaded options Spax, Rothoblaas, Wurth, etc)
- Rods G8.8 M16
- Design required for dowels to carry shear
- Connections can relax once loaded, tightening of rods following loading can reduce connection rotation
- Ductility of connection μ=1.0 for seismic design

MOMENT KNEE CONNECTION, QUICK CONNECT
(MKCQC - 630x90 GL8)

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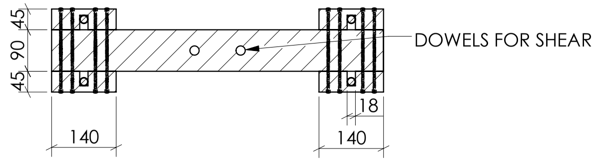
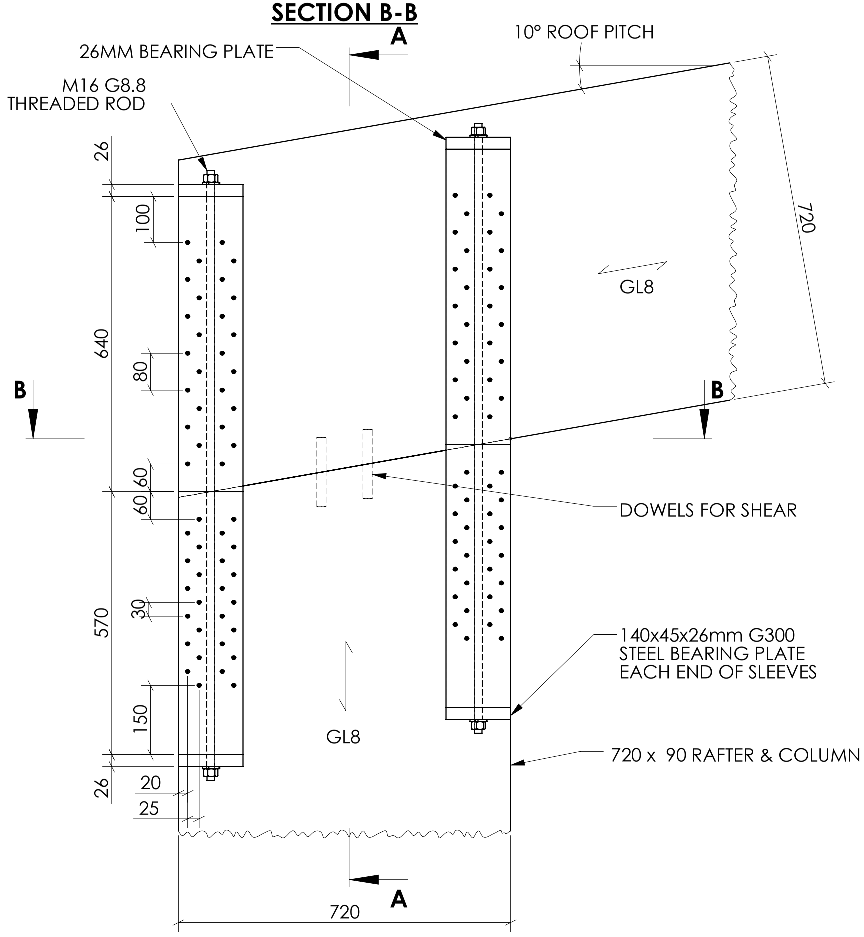
DRAWING NO.
MKCQC - 630x90 GL8

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	5/01/20
B	UPDATED	DJS	5/03/20
C	UPDATED	DJS	1/04/20

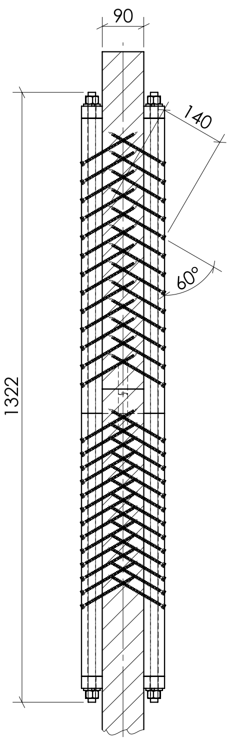
MATERIAL	MEMBER SIZE (mm)	SCREWS FULLY THREADED IN BEAM	SCREWS FULLY THREADED IN COLUMN	MOMENT CAPACITY, (kNm) for K_1=1.0	
				CONNECTION CAPACITY,ØM	MEMBER CAPACITY, ØM
GL8	720 x 90	26/Ø6x140/SLEEVE	26/Ø6x140/SLEEVE	105	118



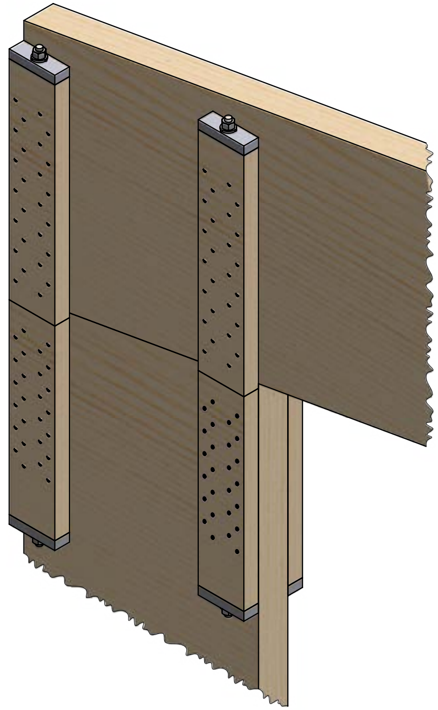
SECTION B-B



ELEVATION



SECTION A-A



NOTES-

- Screws to be fully threaded options Spax, Rothoblaas, Wurth, etc)
- Rods G8.8 M16
- Design required for dowels to carry shear
- Connections can relax once loaded, tightening of rods following loading can reduce connection rotation
- Ductility of connection μ=1.0 for seismic design

MOMENT KNEE CONNECTION, QUICK CONNECT
(MKCQC - 720x90 GL8)

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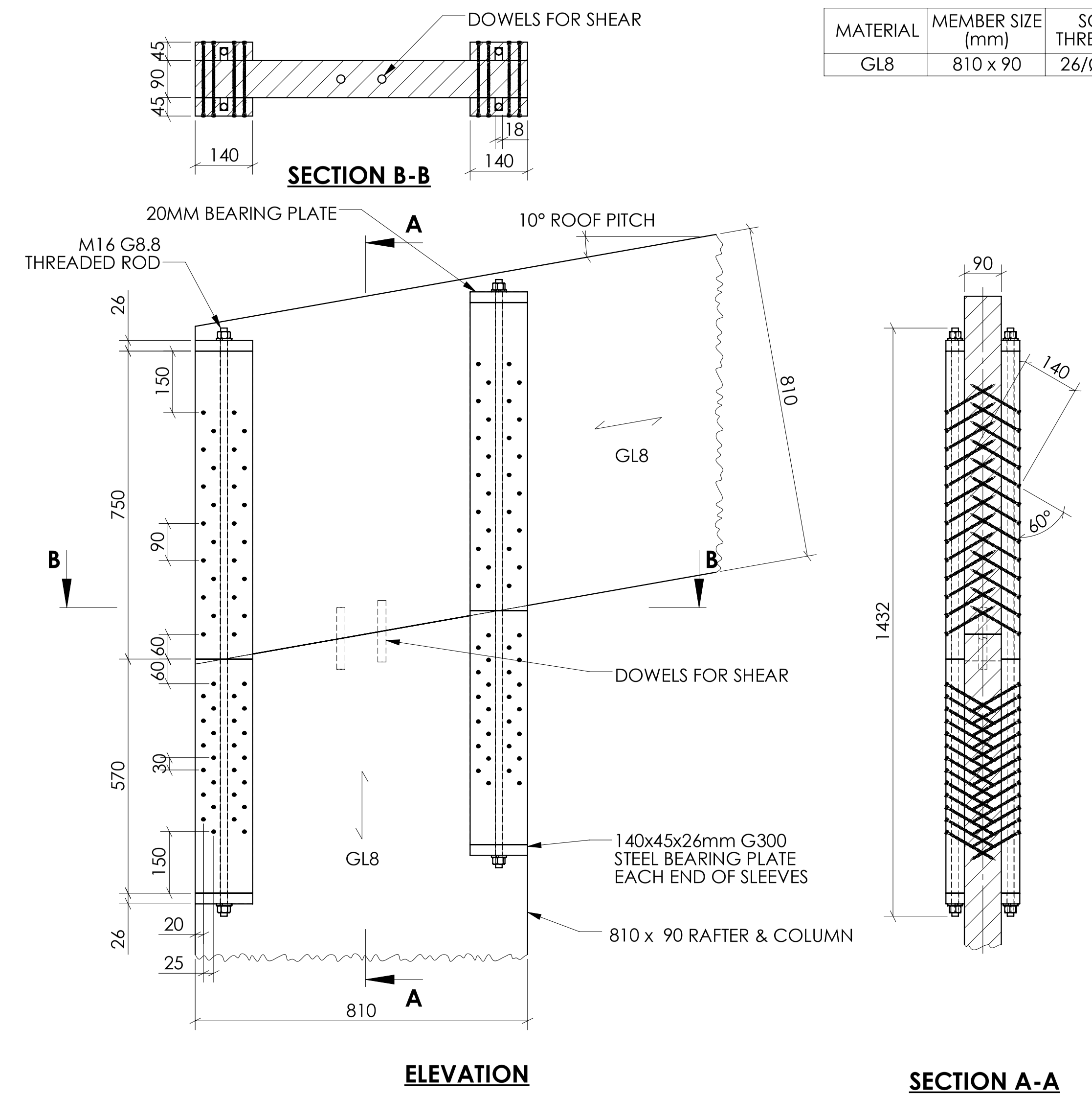
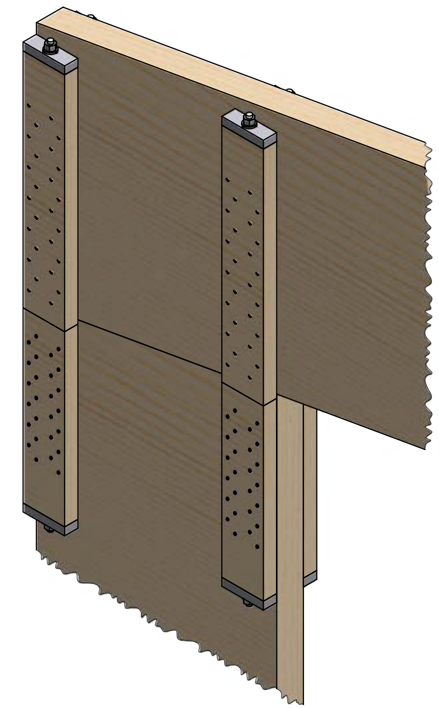
DRAWING NO.
MKCQC - 720x90 GL8

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	5/01/20
B	UPDATED	DJS	6/03/20
C	UPDATED	DJS	1/04/20

MATERIAL	MEMBER SIZE (mm)	SCREWS FULLY THREADED IN BEAM	SCREWS FULLY THREADED IN COLUMN	MOMENT CAPACITY, (kNm) for K_1=1.0 CONNECTION CAPACITY,ØM	MOMENT CAPACITY, (kNm) for K_1=1.0 MEMBER CAPACITY, ØM
GL8	810 x 90	26/Ø6x140/SLEEVE	26/Ø6x140/SLEEVE	120	149





NOTES-

- Screws to be fully threaded options Spax, Rothoblaas, Wurth, etc)
- Rods G8.8 M16
- Design required for dowels to carry shear
- Connections can relax once loaded, tightening of rods following loading can reduce connection rotation
- Ductility of connection μ=1.0 for seismic design

MOMENT KNEE CONNECTION, QUICK CONNECT
(MKCQC - 810x90 GL8)

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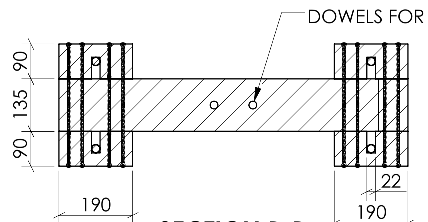
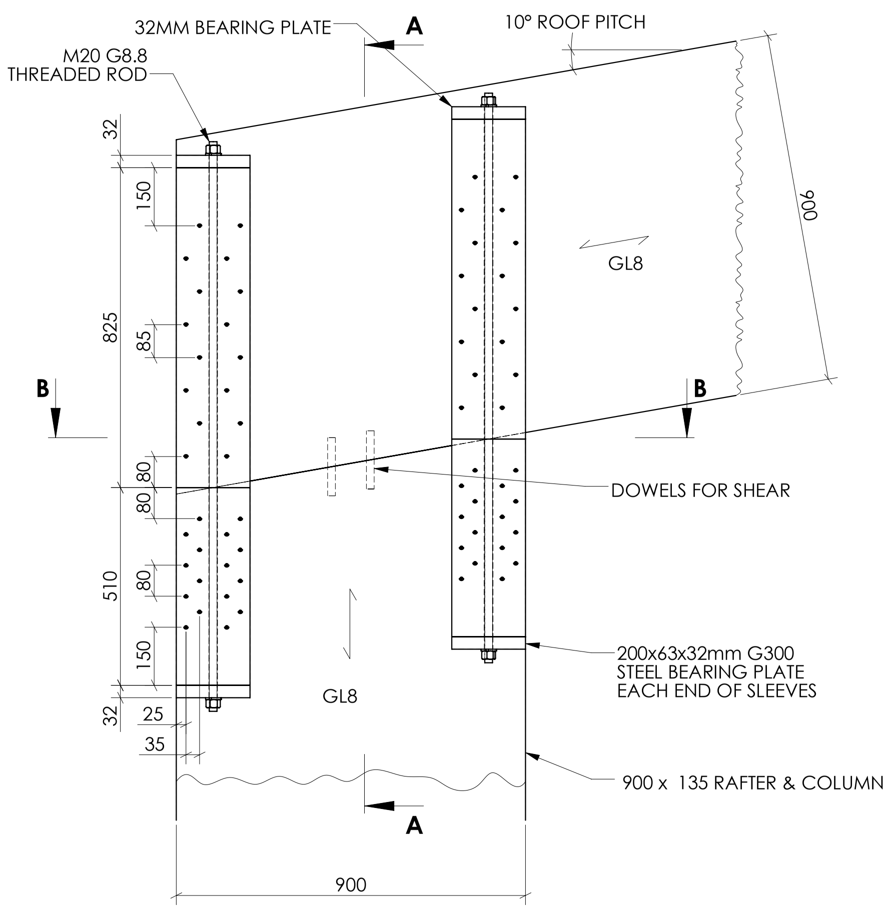
DRAWING NO.
MKCQC - 810x90 GL8

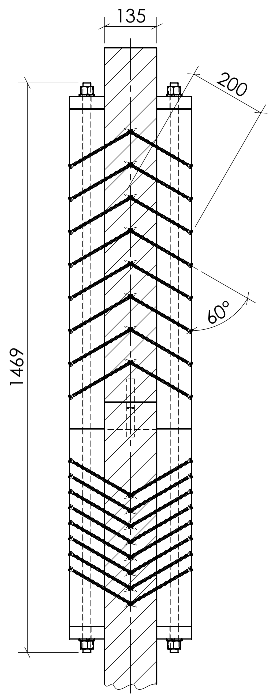
REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	5/01/20
B	UPDATED	DJS	7/03/20
C	UPDATED	DJS	1/04/20



SECTION B-B

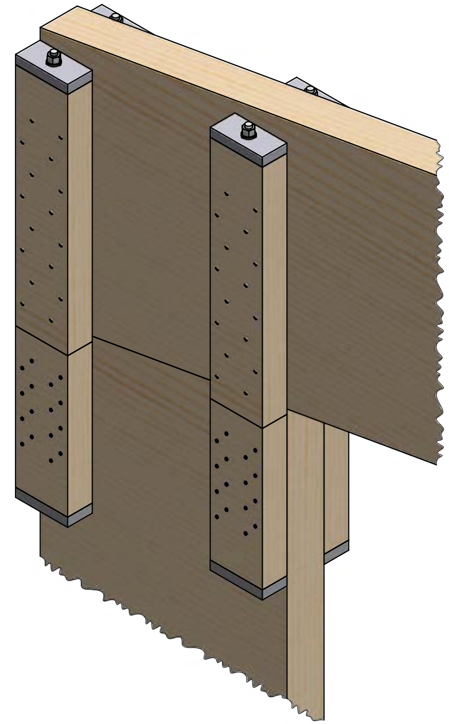


ELEVATION



SECTION A-A

MATERIAL	MEMBER SIZE (mm)	SCREWS FULLY THREADED IN BEAM	SCREWS FULLY THREADED IN COLUMN	MOMENT CAPACITY, (kNm) for K_1=1.0 CONNECTION CAPACITY,ØM	MOMENT CAPACITY, (kNm) for K_1=1.0 MEMBER CAPACITY, ØM
GL8	900 x 135	16/Ø8x200/SLEEVE	16/Ø8x200/SLEEVE	200	227



NOTES-

- Screws to be fully threaded options Spax, Rothoblaas, Wurth, etc)
- Rods G8.8 M20
- Design required for dowels to carry shear
- Connections can relax once loaded, tightening of rods following loading can reduce connection rotation
- Ductility of connection μ=1.0 for seismic design

MOMENT KNEE CONNECTION, QUICK CONNECT
(MKCQC - 900x135 GL8)

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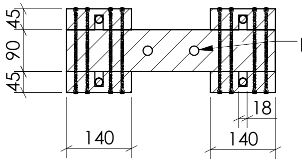
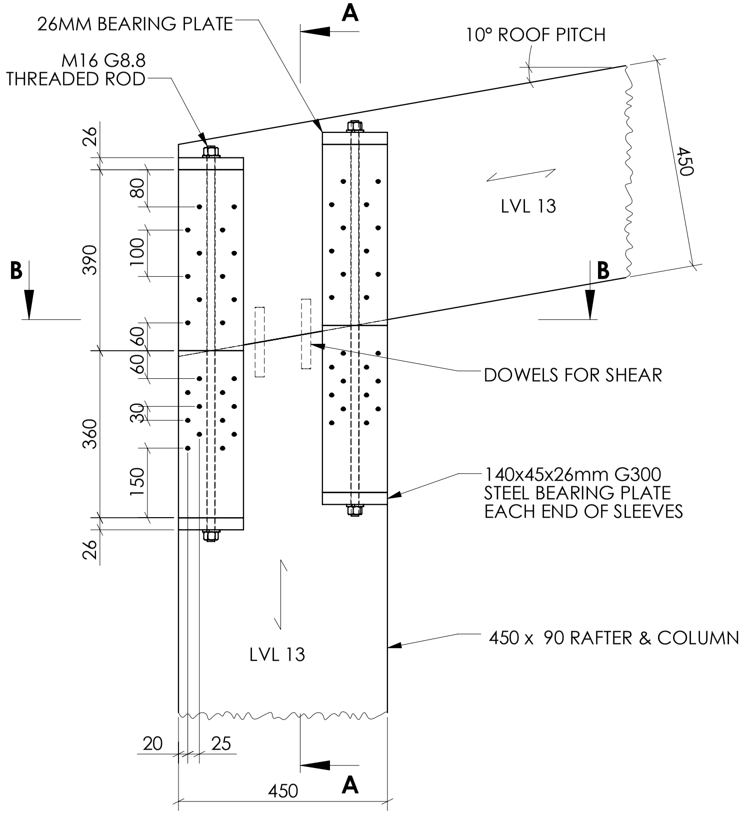
DRAWING NO.
MKCQC - 900x135 GL8

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	5/01/20
B	UPDATED	DJS	7/03/20
C	UPDATED	DJS	1/04/20

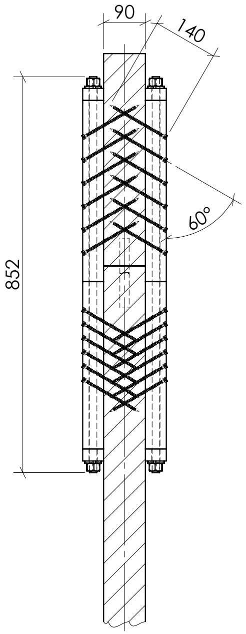
MATERIAL	MEMBER SIZE (mm)	SCREWS FULLY THREADED IN BEAM	SCREWS FULLY THREADED IN COLUMN	MOMENT CAPACITY, (kNm) for K_1=1.0 CONNECTION CAPACITY, ØM	MOMENT CAPACITY, (kNm) for K_1=1.0 MEMBER CAPACITY, ØM
LVL 13	450 x 90	12/Ø6x140/SLEEVE	12/Ø6x140/SLEEVE	45	95



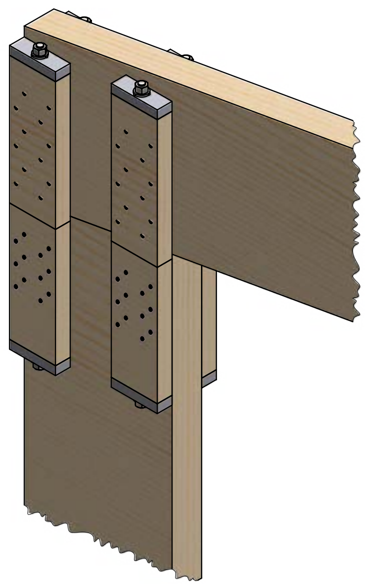
SECTION B-B



ELEVATION



SECTION A-A



NOTES-

- Screws to be fully threaded options Spax, Rothoblaas, Wurth, etc)
- Rods G8.8 M16
- Design required for dowels to carry shear
- Connections can relax once loaded, tightening of rods following loading can reduce connection rotation
- Ductility of connection μ=1.0 for seismic design

MOMENT KNEE CONNECTION, QUICK CONNECT
(MKCQC - 450x90 LVL13)

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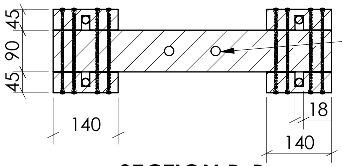
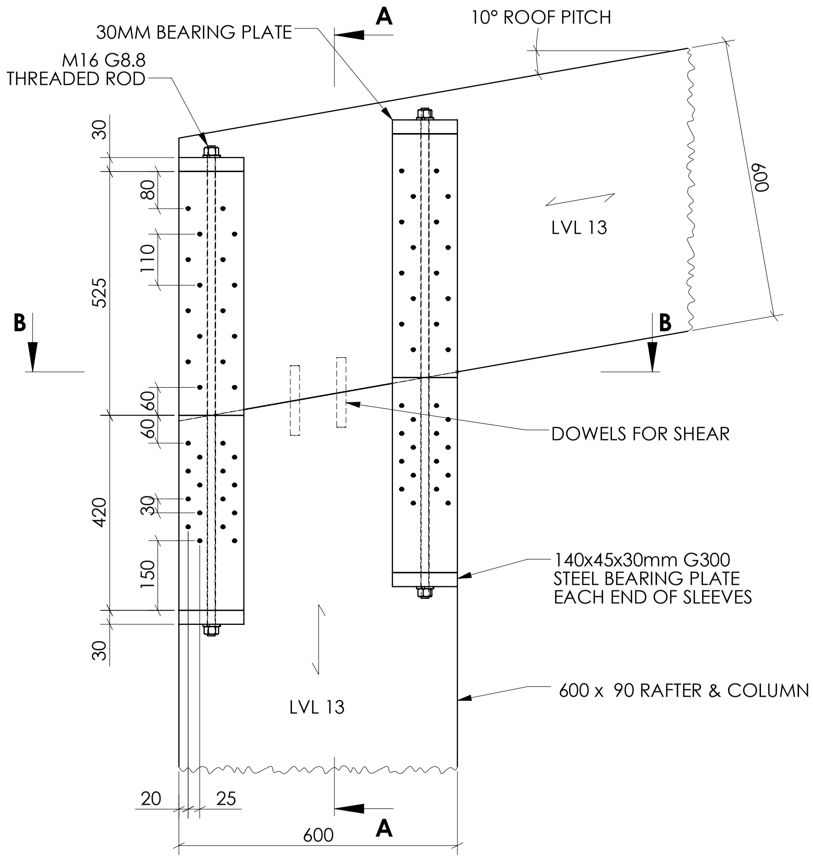
DRAWING NO.
MKCQC - 450x90 LVL13

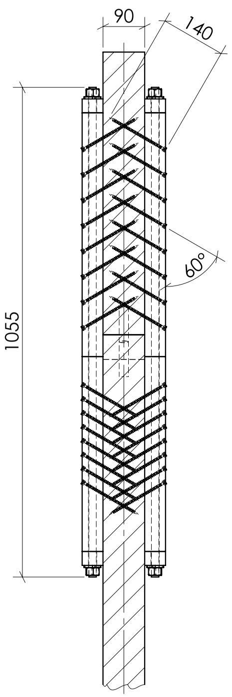
REV	REVISION DESCRIPTION	DRN	DATE
B	UPDATED	DJS	29/04/19
C	UPDATED	DJS	4/01/20
D	UPDATED	DJS	2/03/20
E	UPDATED	DJS	31/03/20



SECTION B-B

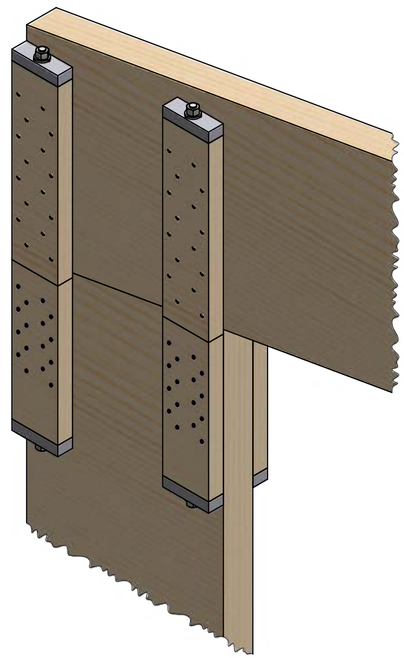


ELEVATION



SECTION A-A

MATERIAL	MEMBER SIZE (mm)	SCREWS FULLY THREADED IN BEAM	SCREWS FULLY THREADED IN COLUMN	MOMENT CAPACITY, (kNm) for K_1=1.0 CONNECTION CAPACITY,ØM	MOMENT CAPACITY, (kNm) for K_1=1.0 MEMBER CAPACITY, ØM
LVL 13	600 x 90	16/Ø6x140/SLEEVE	16/Ø6x140/SLEEVE	90	161



NOTES-

- Screws to be fully threaded options Spax, Rothoblaas, Wurth, etc)
- Rods G8.8 M16
- Design required for dowels to carry shear
- Connections can relax once loaded, tightening of rods following loading can reduce connection rotation
- Ductility of connection μ=1.0 for seismic design

MOMENT KNEE CONNECTION, QUICK CONNECT
(MKCQC - 600x90 LVL13)

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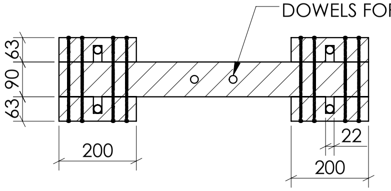
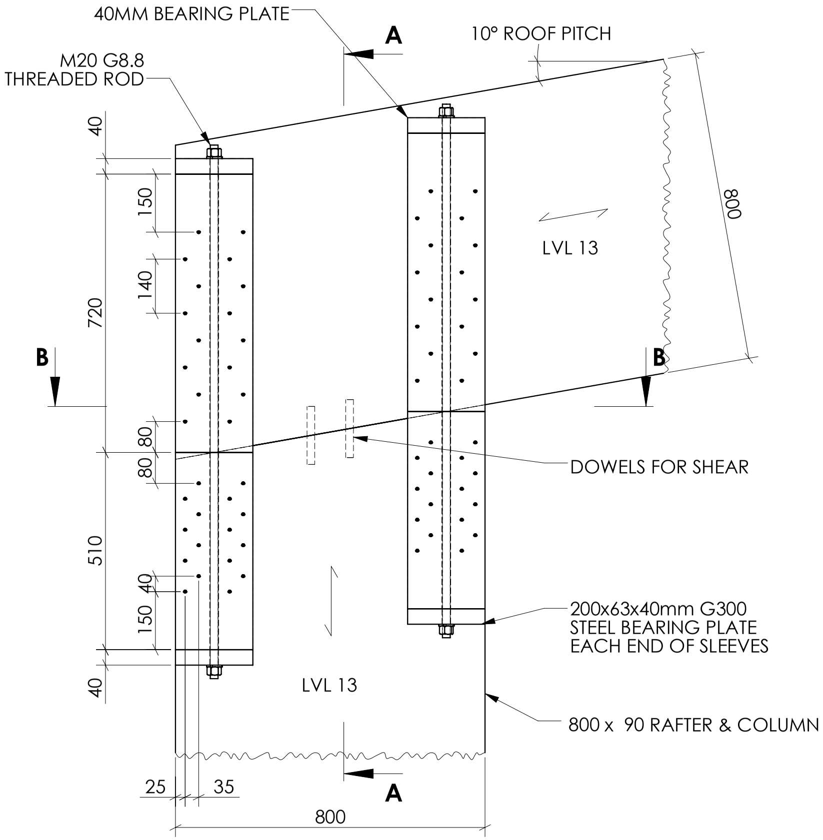
DRAWING NO.
MKCQC - 600x90 LVL13

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	4/03/19
B	UPDATED	DJS	4/01/20
C	UPDATED	DJS	2/03/20
D	UPDATED	DJS	31/03/20

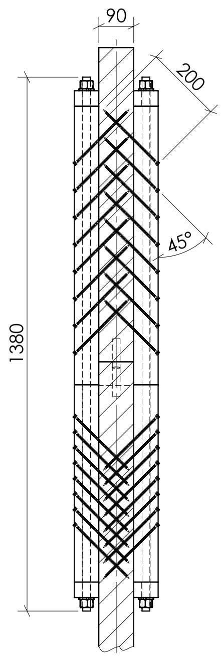
MATERIAL	MEMBER SIZE (mm)	SCREWS FULLY THREADED IN BEAM	SCREWS FULLY THREADED IN COLUMN	MOMENT CAPACITY, (kNm) for K_1=1.0 CONNECTION CAPACITY, ØM	MEMBER CAPACITY, ØM
LVL 13	800 x 90	16/Ø8x200/SLEEVE	16/Ø8x200/SLEEVE	190	272



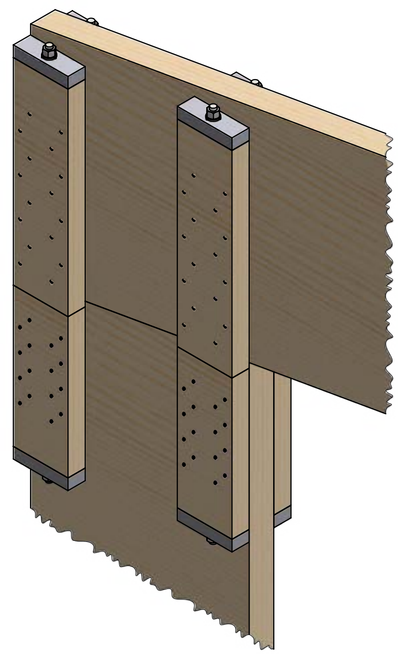
SECTION B-B



ELEVATION



SECTION A-A



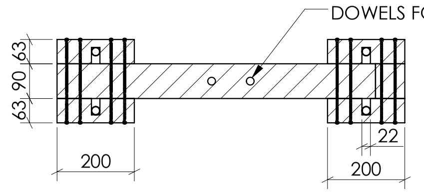
NOTES-

- Screws to be fully threaded options Spax, Rothoblaas, Wurth, etc)
- Rods G8.8 M20
- Design required for dowels to carry shear
- Connections can relax once loaded, tightening of rods following loading can reduce connection rotation
- Ductility of connection μ=1.0 for seismic design

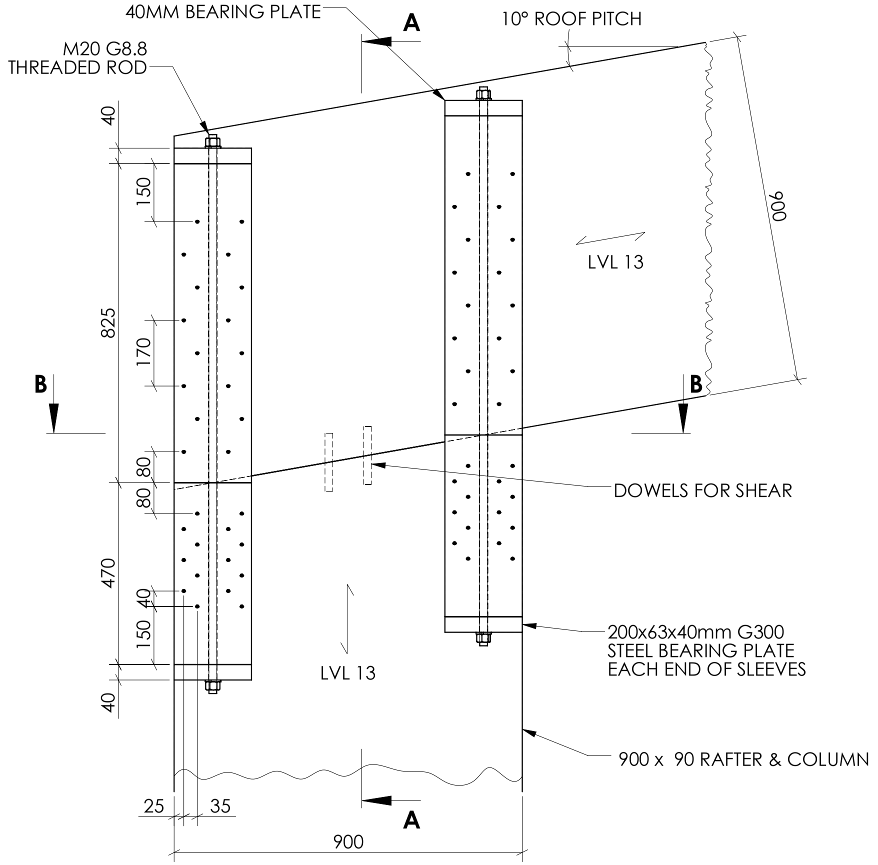
MOMENT KNEE CONNECTION, QUICK CONNECT (MKCQC - 800x90 LVL13)	www.nzwood.co.nz design@nzwood.co.nz	DRAWING NO. MKCQC - 800x90 LVL13

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	4/03/19
B	UPDATED	DJS	4/01/20
C	UPDATED	DJS	2/03/20
D	UPDATED	DJS	31/03/20

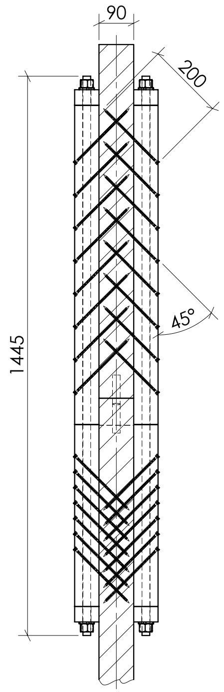
MATERIAL	MEMBER SIZE (mm)	SCREWS FULLY THREADED IN BEAM	SCREWS FULLY THREADED IN COLUMN	MOMENT CAPACITY, (kNm) for K_1=1.0 CONNECTION CAPACITY,ØM	MEMBER CAPACITY, ØM
LVL 13	900 x 90	16/Ø8x200/SLEEVE	14/Ø8x200/SLEEVE	220	338



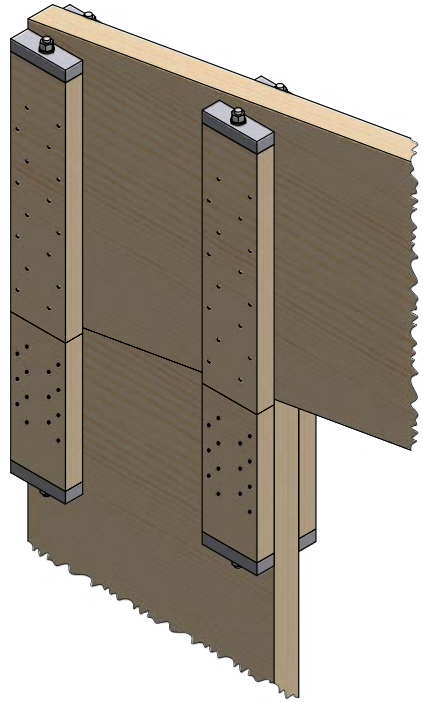
SECTION B-B



ELEVATION



SECTION A-A



NOTES-

- Screws to be fully threaded options Spax, Rothoblaas, Wurth, etc)
- Rods G8.8 M20
- Design required for dowels to carry shear
- Connections can relax once loaded, tightening of rods following loading can reduce connection rotation
- Ductility of connection μ=1.0 for seismic design

MOMENT KNEE CONNECTION, QUICK CONNECT (MKCQC - 900x90 LVL13)

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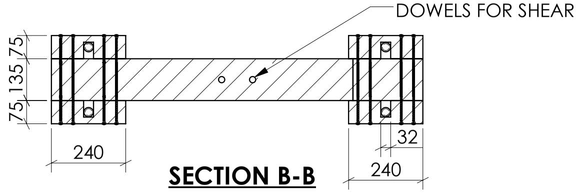
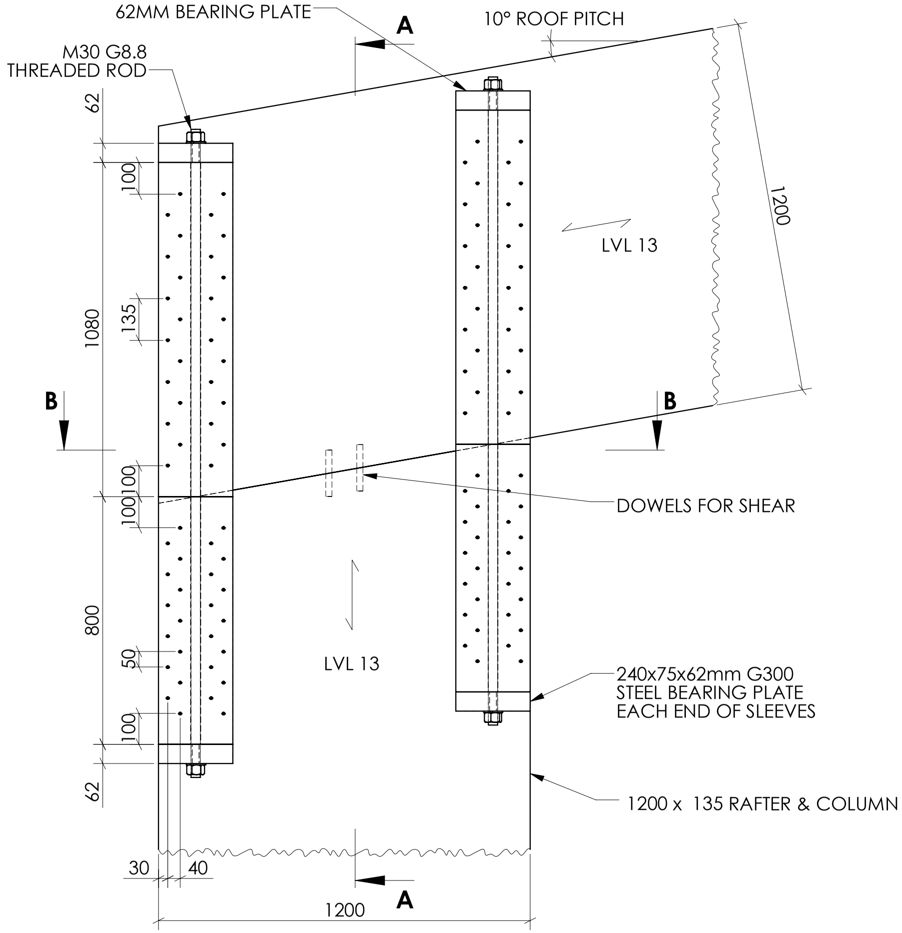
DRAWING NO.
MKCQC - 900x90 LVL13

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	4/03/19
B	UPDATED	DJS	4/01/20
C	UPDATED	DJS	2/03/20
D	UPDATED	DJS	31/03/20

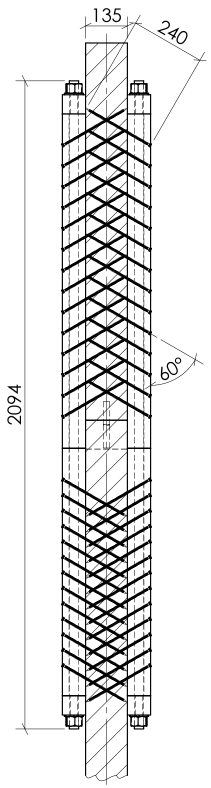
MATERIAL	MEMBER SIZE (mm)	SCREWS FULLY THREADED IN BEAM	SCREWS FULLY THREADED IN COLUMN	MOMENT CAPACITY, (kNm) for K_1=1.0 CONNECTION CAPACITY,ØM	MEMBER CAPACITY, ØM
LVL 13	1200 x 135	28/Ø10x240/SLEEVE	28/Ø10x240/SLEEVE	700	859



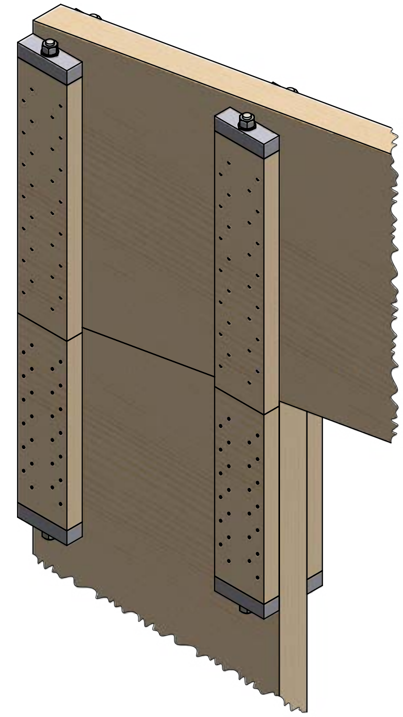
SECTION B-B



ELEVATION



SECTION A-A



NOTES-

- Screws to be fully threaded options Spax, Rothoblaas, Wurth, etc)
- Rods G8.8 M30
- Design required for dowels to carry shear
- Connections can relax once loaded, tightening of rods following loading can reduce connection rotation
- Ductility of connection μ=1.0 for seismic design

MOMENT KNEE CONNECTION, QUICK CONNECT (MKCQC - 1200x135 LVL13)

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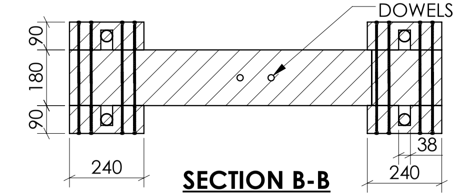
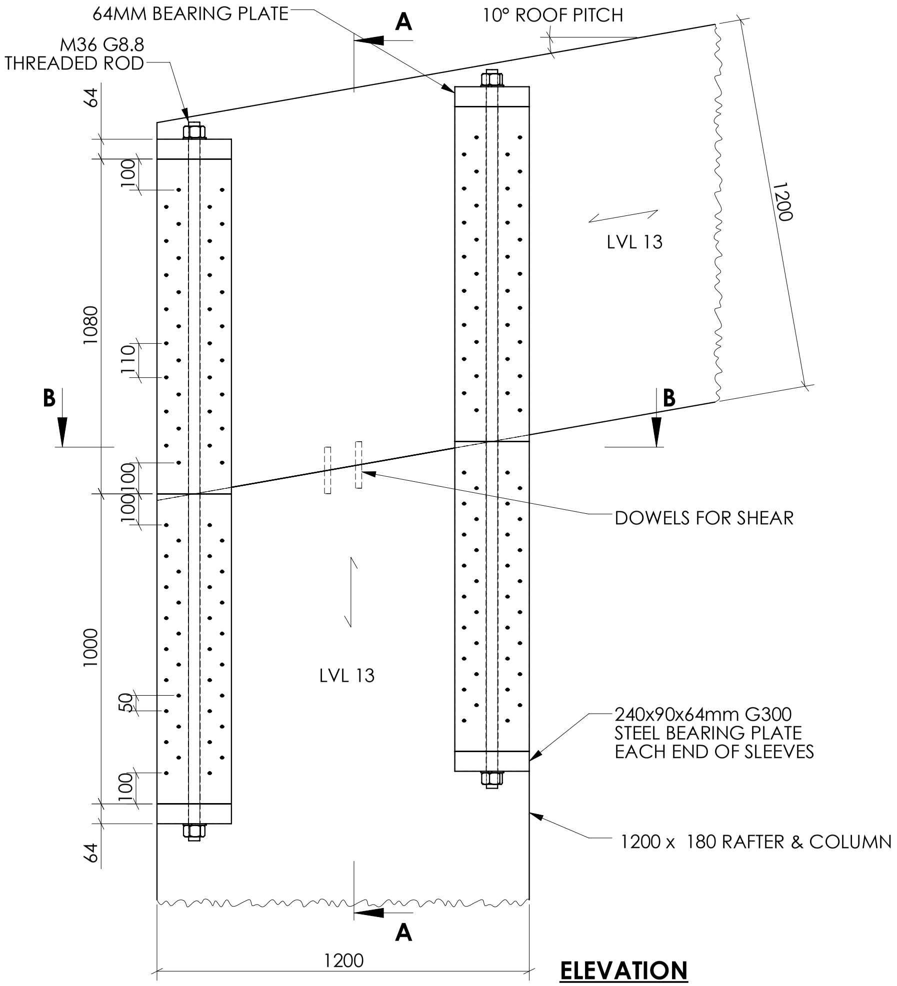
DRAWING NO.	REV	REVISION DESCRIPTION	DRN	DATE
MKCQC - 1200x135 LVL13	A	ISSUED FOR INFORMATION	DJS	4/03/19
	B	UPDATED	DJS	5/01/20
	C	UPDATED	DJS	2/03/20
	D	UPDATED	DJS	31/03/20

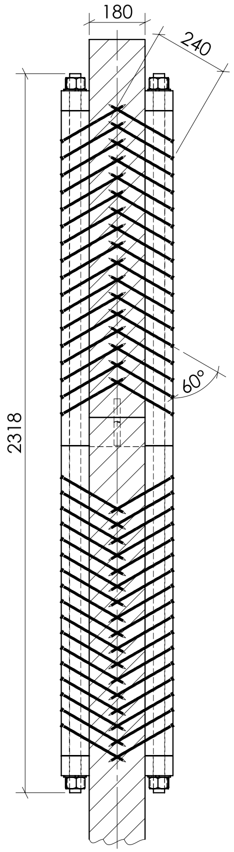
MATERIAL	MEMBER SIZE (mm)	SCREWS FULLY THREADED IN BEAM	SCREWS FULLY THREADED IN COLUMN	MOMENT CAPACITY, (kNm) for K_1=1.0 CONNECTION CAPACITY,ØM	MOMENT CAPACITY, (kNm) for K_1=1.0 MEMBER CAPACITY, ØM
LVL 13	1200 x 180	34/Ø10x240/SLEEVE	34/Ø10x240/SLEEVE	835	1146



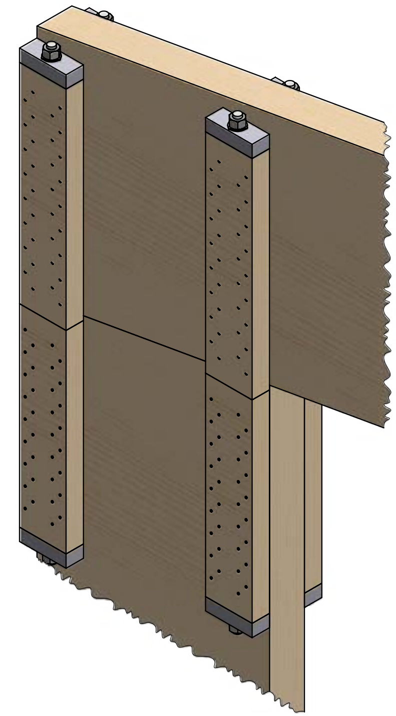
SECTION B-B



ELEVATION



SECTION A-A

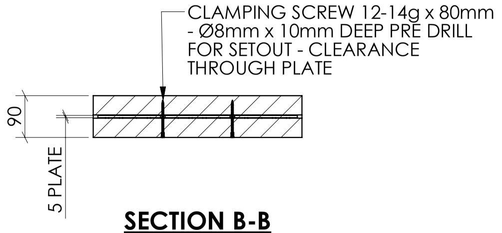


NOTES-

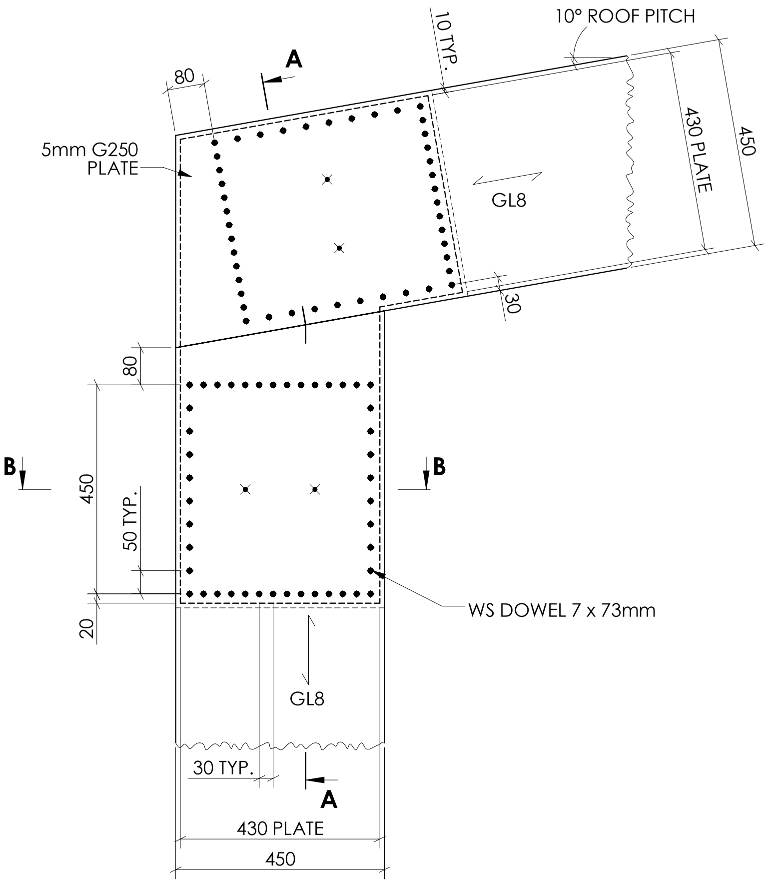
- Screws to be fully threaded options Spax, Rothoblaas, Wurth, etc)
- Rods G8.8 M36
- Design required for dowels to carry shear
- Connections can relax once loaded, tightening of rods following loading can reduce connection rotation
- Ductility of connection μ=1.0 for seismic design

MOMENT KNEE CONNECTION, QUICK CONNECT (MKCQC - 1200x180 LVL13)	NZ Wood	www.nzwood.co.nz design@nzwood.co.nz	DRAWING NO. MKCQC - 1200x180 LVL13	REV	REVISION DESCRIPTION	DRN	DATE
				A	ISSUED FOR INFORMATION	DJS	4/03/19
				B	UPDATED	DJS	5/01/20
				C	UPDATED	DJS	3/03/20
				D	UPDATED	DJS	1/04/20

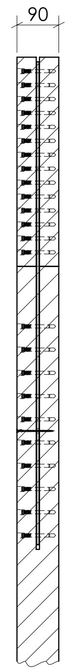
MATERIAL	MEMBER SIZE (mm)	G250 PLATE THICKNESS (mm)	G250 PLATE WIDTH (mm)	TOTAL NUMBER OF WS DOWELS TO EACH GROUP	MOMENT CAPACITY, (kNm) for K_1=1.0		
					DOWEL CAPACITY,ØM	PLATE YIELD CAPACITY, ØM	MEMBER CAPACITY, ØM
GL8	450 x 90	5	430	44	40	35	47



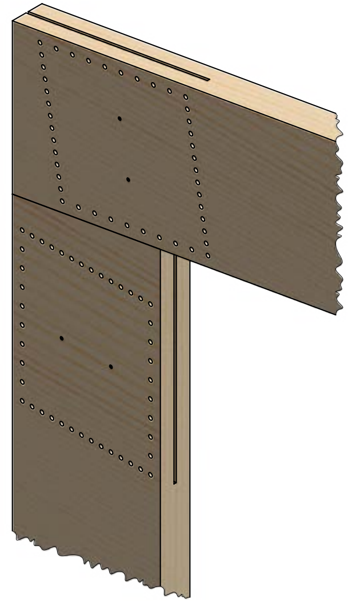
SECTION B-B



ELEVATION



SECTION A-A



NOTES-

- Predrill head diameter Ø11x15mm hole, to position dowel and prevent splitting near the head.
- WS Dowels self drills into timber and steel. Use a 1500rpm driver.
- Use G250 steel plate to allow cutting heads to penetrate.
- Capacities of the WS Dowels are taken from WS-01 (Rothoblaas) Technical data sheet using an LVL density of 480kg/m3.
- Ductility of connection μ=1.25-1.5 for seismic design.

MOMENT KNEE CONNECTION, WS DOWEL AND STEEL PLATE (MKCWSS - 450x90 GL8)

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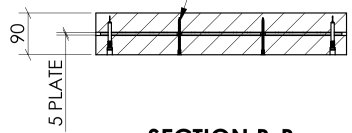
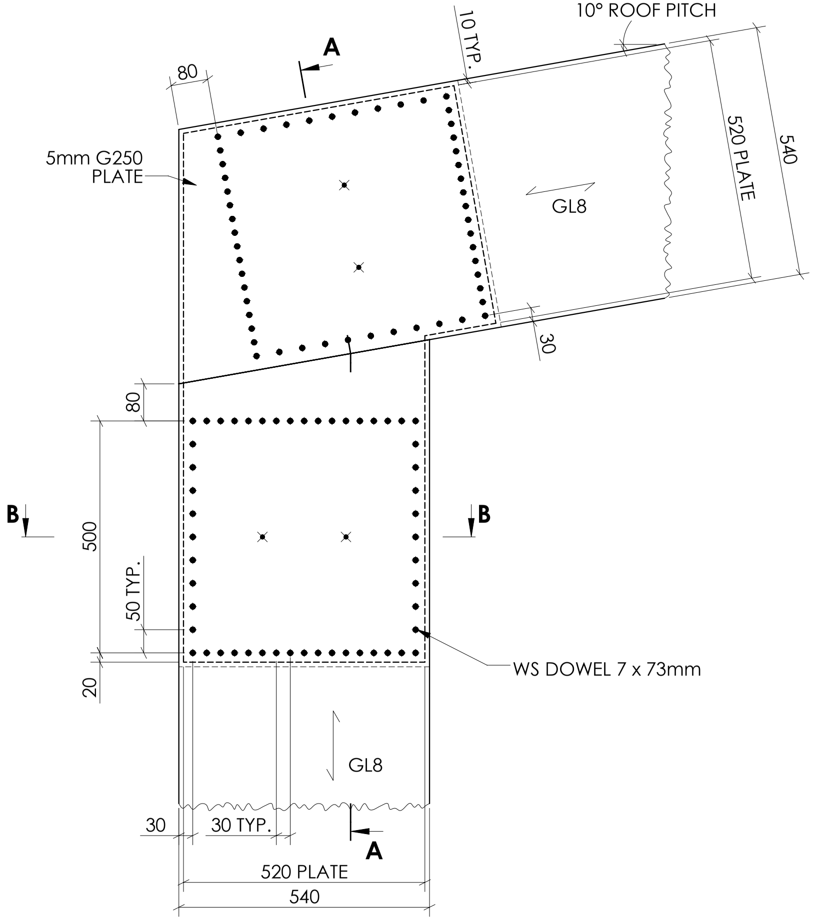
DRAWING NO.
MKCWSS - 450x90 GL8

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	5/01/20
B	UPDATED	DJS	24/02/20

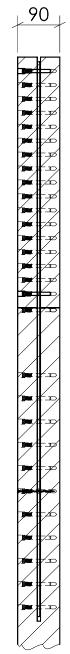
MATERIAL	MEMBER SIZE (mm)	G250 PLATE THICKNESS (mm)	G250 PLATE WIDTH (mm)	TOTAL NUMBER OF WS DOWELS TO EACH GROUP	MOMENT CAPACITY, (kNm) for K_1=1.0		
					DOWEL CAPACITY,ØM	PLATE YIELD CAPACITY, ØM	MEMBER CAPACITY, ØM
GL8	540 x 90	5	520	52	54	51	68



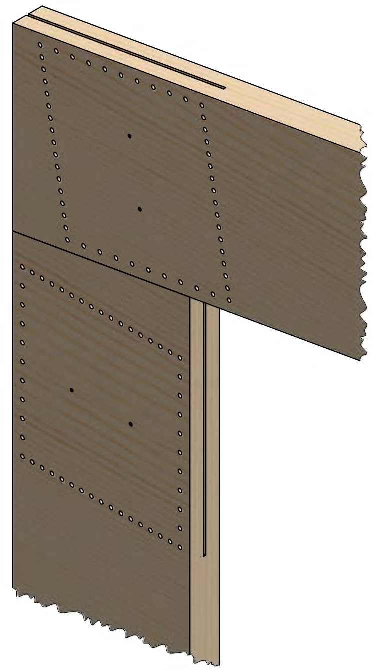
SECTION B-B



ELEVATION



SECTION A-A



NOTES-

- Predrill head diameter Ø11x15mm hole, to position dowel and prevent splitting near the head.
- WS Dowels self drills into timber and steel. Use a 1500rpm driver.
- Use G250 steel plate to allow cutting heads to penetrate.
- Capacities of the WS Dowels are taken from WS-01 (Rothoblaas) Technical data sheet using an LVL density of 480kg/m3.
- Ductility of connection μ=1.25-1.5 for seismic design.

MOMENT KNEE CONNECTION, WS DOWEL AND STEEL PLATE (MKCWSS - 540x90 GL8)

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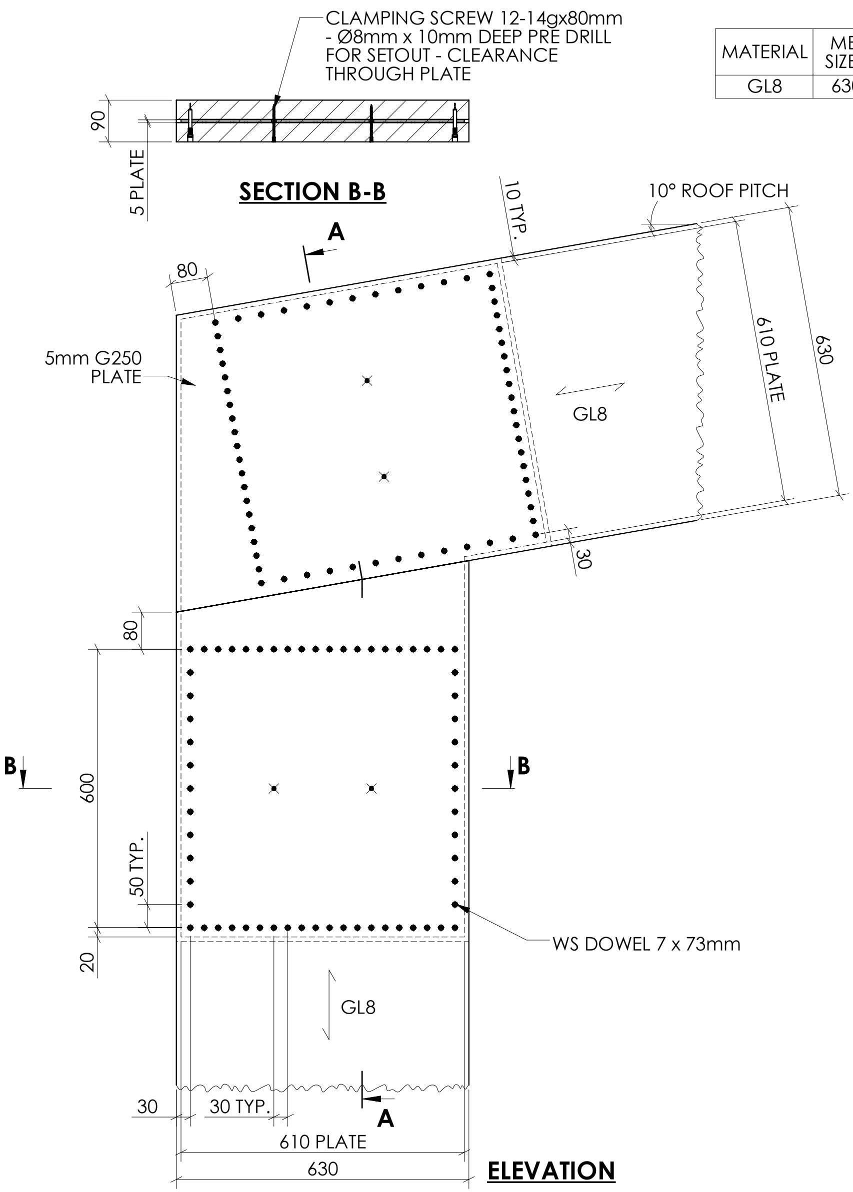
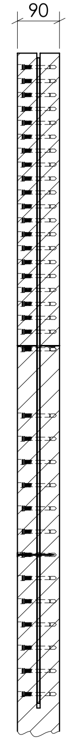
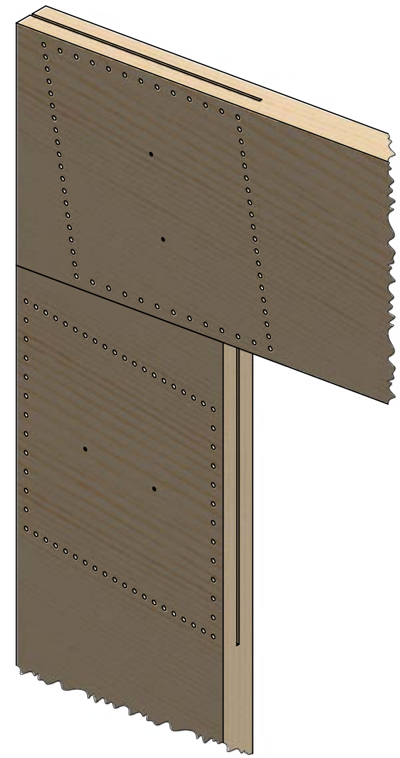
DRAWING NO.
MKCWSS - 540x90 GL8

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	5/01/20
B	UPDATED	DJS	24/02/20



MATERIAL	MEMBER SIZE (mm)	G250 PLATE THICKNESS (mm)	G250 PLATE WIDTH (mm)	TOTAL NUMBER OF WS DOWELS TO EACH GROUP	MOMENT CAPACITY, (kNm) for K_1=1.0 DOWEL CAPACITY,ØM	PLATE YIELD CAPACITY, ØM	MEMBER CAPACITY, ØM
GL8	630 x 90	5	610	62	77	70	93





NOTES-

- Predrill head diameter Ø11x15mm hole, to position dowel and prevent splitting near the head.
- WS Dowels self drills into timber and steel. Use a 1500rpm driver.
- Use G250 steel plate to allow cutting heads to penetrate.
- Capacities of the WS Dowels are taken from WS-01 (Rothoblaas) Technical data sheet using an LVL density of 480kg/m3.
- Ductility of connection μ=1.25-1.5 for seismic design.

MOMENT KNEE CONNECTION, WS DOWEL AND STEEL PLATE (MKCWSS - 630x90 GL8)

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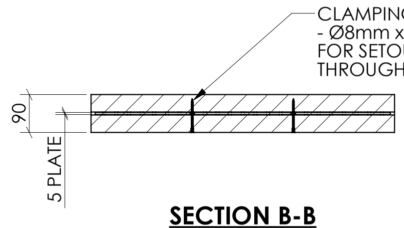
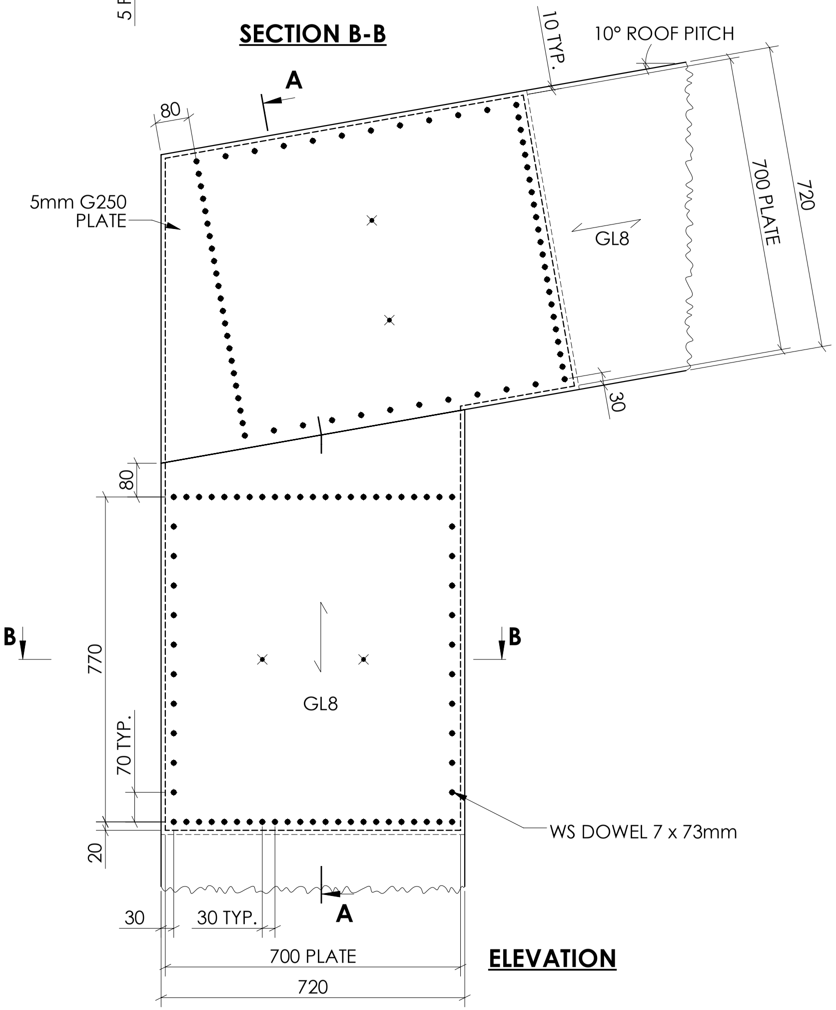
DRAWING NO.	REV	REVISION DESCRIPTION	DRN	DATE
MKCWSS - 630x90 GL8	A	ISSUED FOR INFORMATION	DJS	5/01/20
	B	UPDATED	DJS	24/02/20

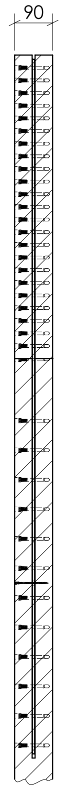
MATERIAL	MEMBER SIZE (mm)	G250 PLATE THICKNESS (mm)	G250 PLATE WIDTH (mm)	TOTAL NUMBER OF WS DOWELS TO EACH GROUP	MOMENT CAPACITY, (kNm) for K_1=1.0		
					DOWEL CAPACITY,ØM	PLATE YIELD CAPACITY, ØM	MEMBER CAPACITY, ØM
GL8	720 x 90	5	700	66	102	92	121



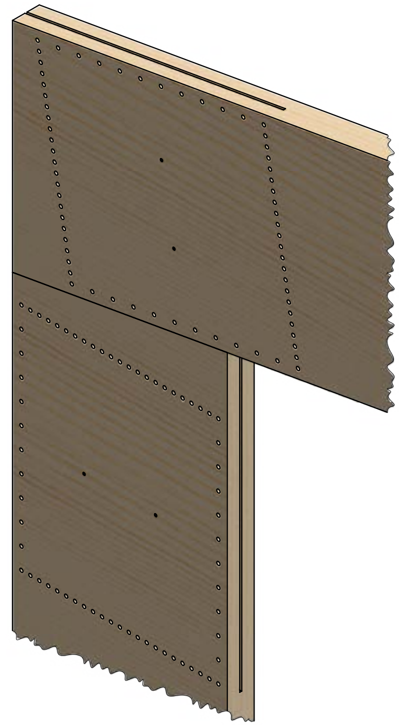
SECTION B-B



ELEVATION



SECTION A-A



NOTES-

- Predrill head diameter Ø11x15mm hole, to position dowel and prevent splitting near the head.
- WS Dowels self drills into timber and steel. Use a 1500rpm driver.
- Use G250 steel plate to allow cutting heads to penetrate.
- Capacities of the WS Dowels are taken from WS-01 (Rothoblaas) Technical data sheet using an LVL density of 480kg/m3.
- Ductility of connection μ=1.25-1.5 for seismic design.

MOMENT KNEE CONNECTION, WS DOWEL AND STEEL PLATE (MKCWSS - 720x90 GL8)

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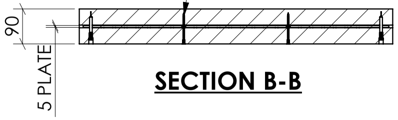
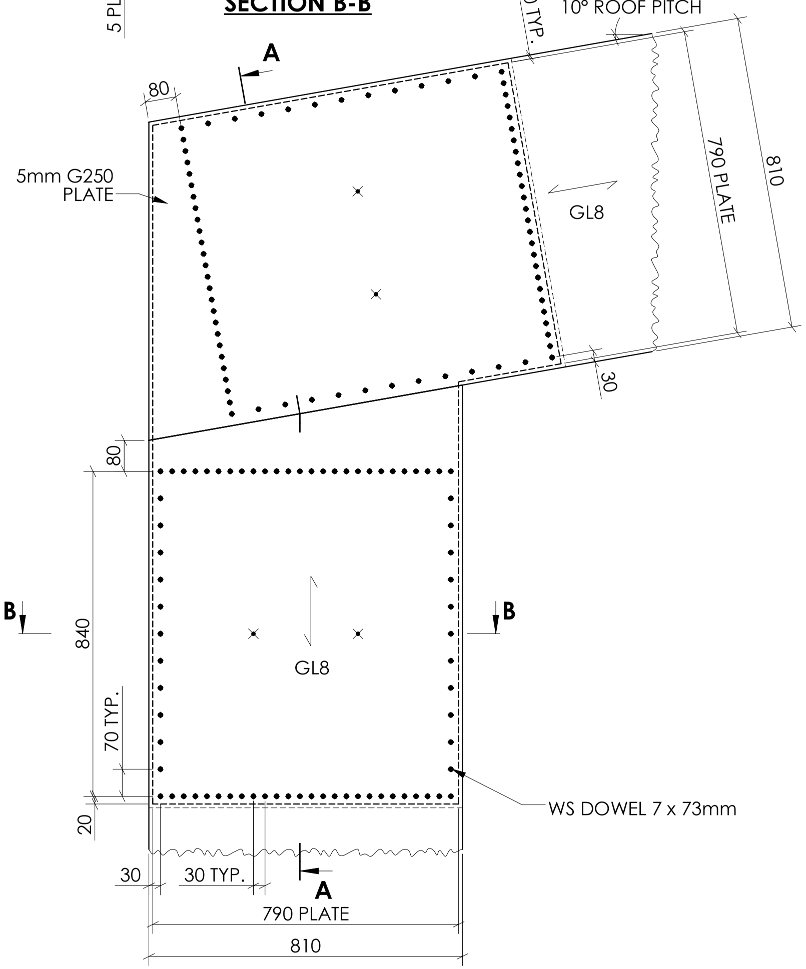
DRAWING NO.
MKCWSS - 720x90 GL8

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	5/01/20
B	UPDATED	DJS	24/02/20

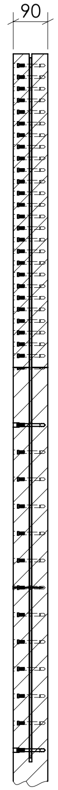
MATERIAL	MEMBER SIZE (mm)	G250 PLATE THICKNESS (mm)	G250 PLATE WIDTH (mm)	TOTAL NUMBER OF WS DOWELS TO EACH GROUP	MOMENT CAPACITY, (kNm) for K_1=1.0 DOWEL CAPACITY,ØM	MOMENT CAPACITY, (kNm) for K_1=1.0 PLATE YIELD CAPACITY, ØM	MOMENT CAPACITY, (kNm) for K_1=1.0 MEMBER CAPACITY, ØM
GL8	810 x 90	5	790	74	127	117	153



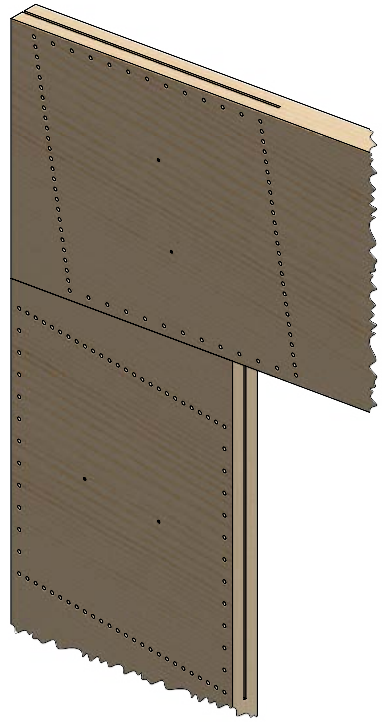
SECTION B-B



ELEVATION



SECTION A-A



NOTES-

- Predrill head diameter Ø11x15mm hole, to position dowel and prevent splitting near the head.
- WS Dowels self drills into timber and steel. Use a 1500rpm driver.
- Use G250 steel plate to allow cutting heads to penetrate.
- Capacities of the WS Dowels are taken from WS-01 (Rothoblaas) Technical data sheet using an LVL density of 480kg/m3.
- Ductility of connection μ=1.25-1.5 for seismic design.

MOMENT KNEE CONNECTION, WS DOWEL AND STEEL PLATE (MKCWSS - 810x90 GL8)

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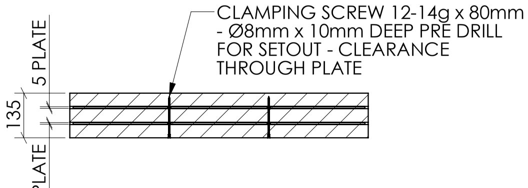
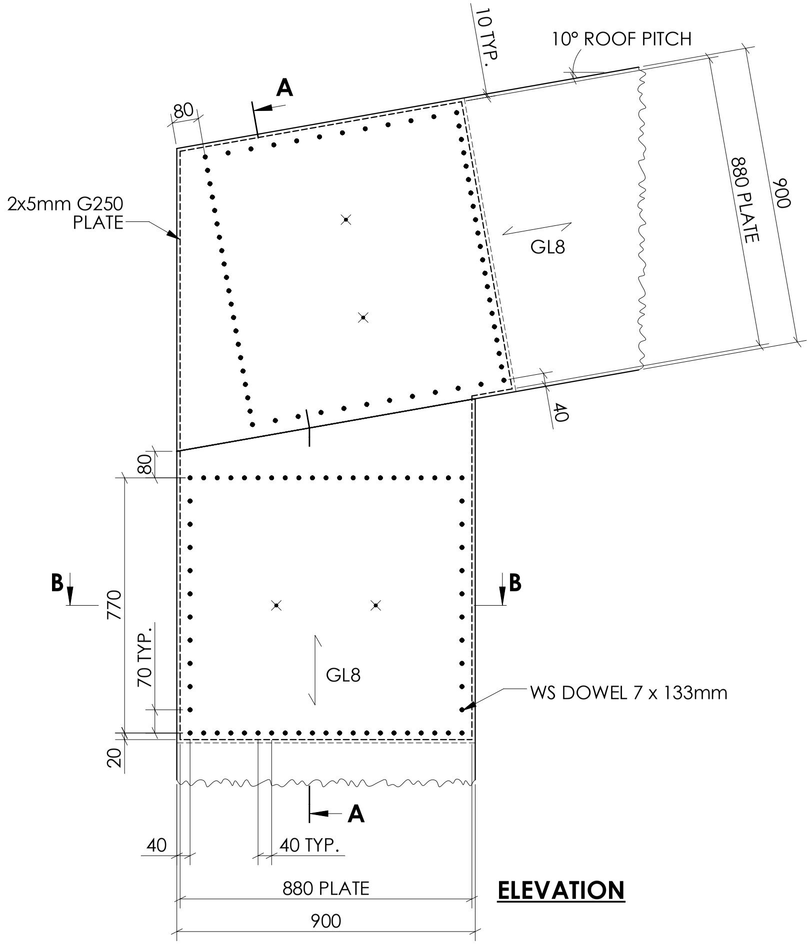
DRAWING NO.
MKCWSS - 810x90 GL8

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	5/01/20
B	UPDATED	DJS	25/02/20

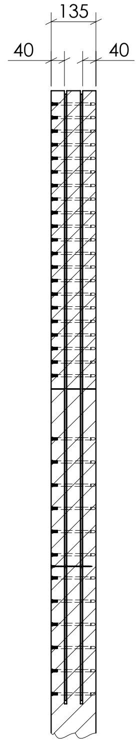


SECTION B-B

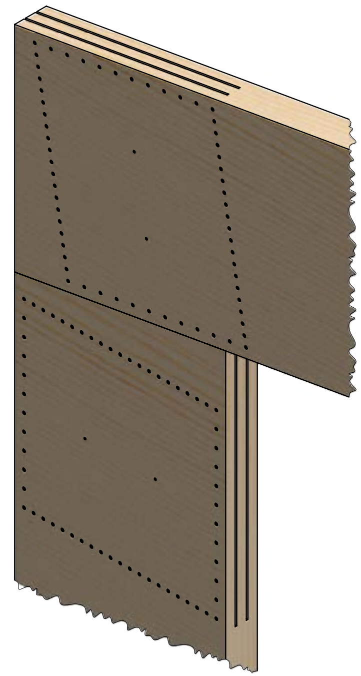
MATERIAL	MEMBER SIZE (mm)	G250 PLATE THICKNESS (mm)	G250 PLATE WIDTH (mm)	TOTAL NUMBER OF WS DOWELS TO EACH GROUP	MOMENT CAPACITY, (kNm) for K_1=1.0 DOWEL CAPACITY,ØM	PLATE YIELD CAPACITY, ØM	MEMBER CAPACITY, ØM
GL8	900 x 135	2/ 5	880	68	313	290	275



ELEVATION



SECTION A-A



NOTES-

- Predrill head diameter Ø11x5mm hole, to position dowel and prevent splitting near the head.
- WS Dowels self drills into timber and steel. Use a 1500rpm driver.
- Use G250 steel plate to allow cutting heads to penetrate.
- Capacities of the WS Dowels are taken from WS-01 (Rothoblaas) Technical data sheet using an LVL density of 480kg/m3.
- Ductility of connection μ=1.25-1.5 for seismic design.

MOMENT KNEE CONNECTION, WS DOWEL AND STEEL PLATE (MKCWSS - 900x135 GL8)

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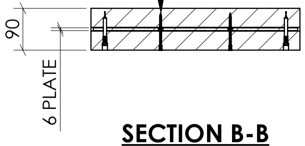
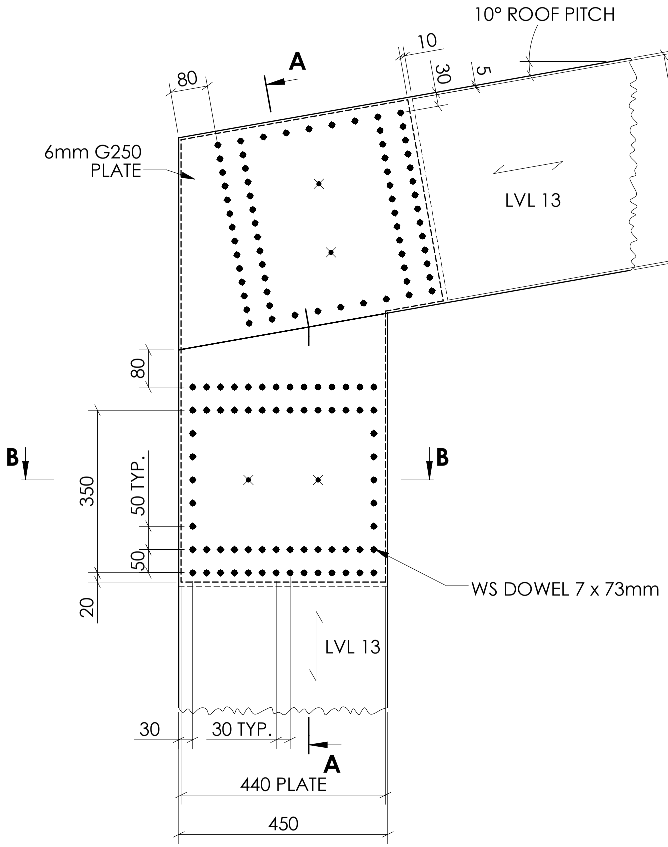
DRAWING NO.: MKCWSS - 900x135 GL8

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	5/01/20
B	UPDATED	DJS	11/02/20

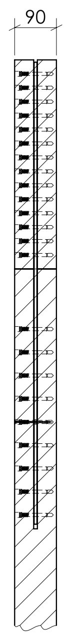
MATERIAL	MEMBER SIZE (mm)	G250 PLATE THICKNESS (mm)	G250 PLATE WIDTH (mm)	TOTAL NUMBER OF WS DOWELS TO EACH GROUP	MOMENT CAPACITY, (kNm) for K_1=1.0		
					DOWEL CAPACITY,ØM	PLATE YIELD CAPACITY, ØM	MEMBER CAPACITY, ØM
LVL 13	450 x 90	6	440	66	49	44	86



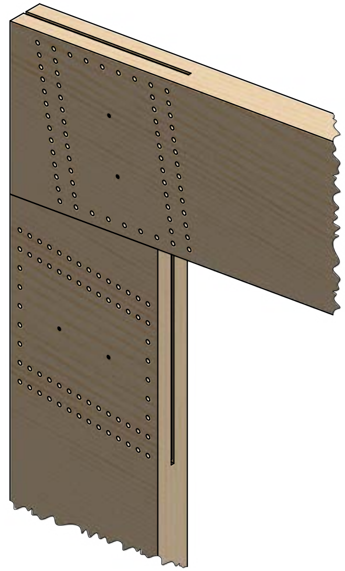
SECTION B-B



ELEVATION



SECTION A-A



NOTES-

- Predrill head diameter Ø11x15mm hole, to position dowel and prevent splitting near the head.
- WS Dowels self drills into timber and steel. Use a 1500rpm driver.
- Use G250 steel plate to allow cutting heads to penetrate.
- Capacities of the WS Dowels are taken from WS-01 (Rothoblaas) Technical data sheet using an LVL density of 480kg/m3.
- Ductility of connection μ=1.25-1.5 for seismic design.

MOMENT KNEE CONNECTION, WS DOWEL AND STEEL PLATE (MKCWSS - 450x90 LVL13)

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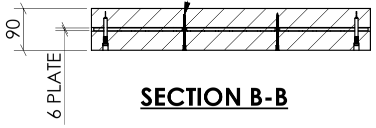
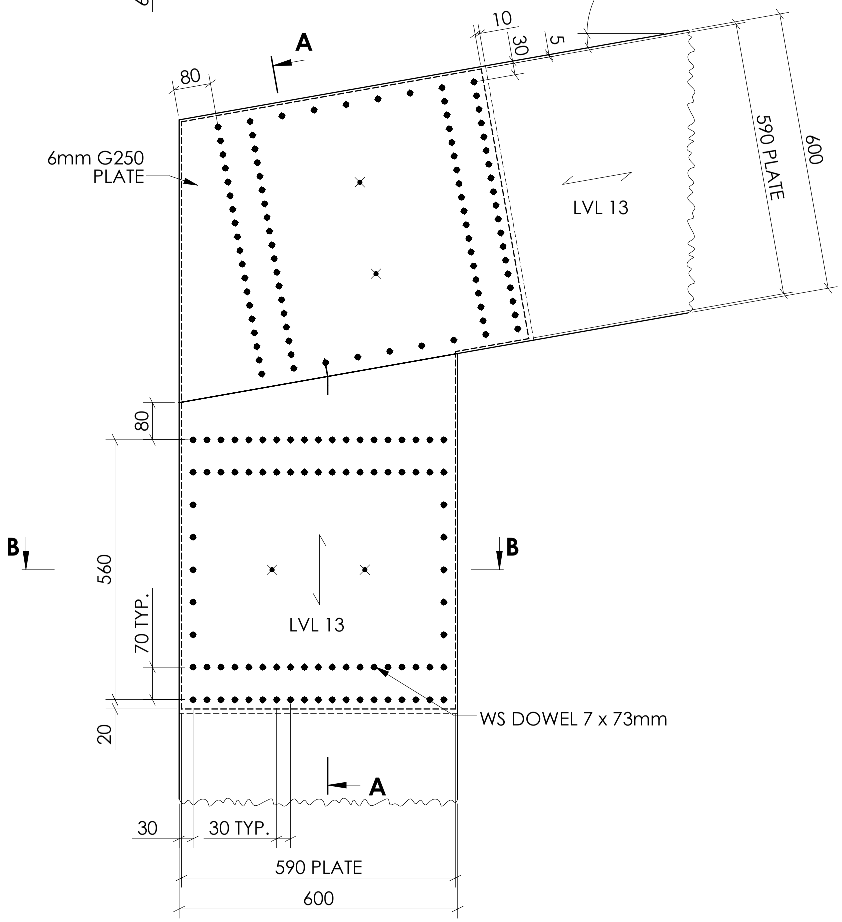
DRAWING NO.
MKCWSS - 450x90 LVL13

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	4/03/19
B	UPDATED	DJS	29/04/19
C	UPDATED	DJS	5/01/20
D	UPDATED	DJS	26/02/20

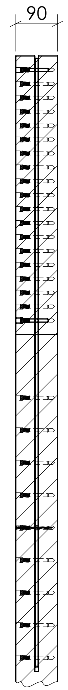
MATERIAL	MEMBER SIZE (mm)	G250 PLATE THICKNESS (mm)	G250 PLATE WIDTH (mm)	TOTAL NUMBER OF WS DOWELS TO EACH GROUP	MOMENT CAPACITY, (kNm) for K_1=1.0 DOWEL CAPACITY,ØM	MOMENT CAPACITY, (kNm) for K_1=1.0 PLATE YIELD CAPACITY, ØM	MOMENT CAPACITY, (kNm) for K_1=1.0 MEMBER CAPACITY, ØM
LVL 13	600 x 90	6	590	88	103	78	146



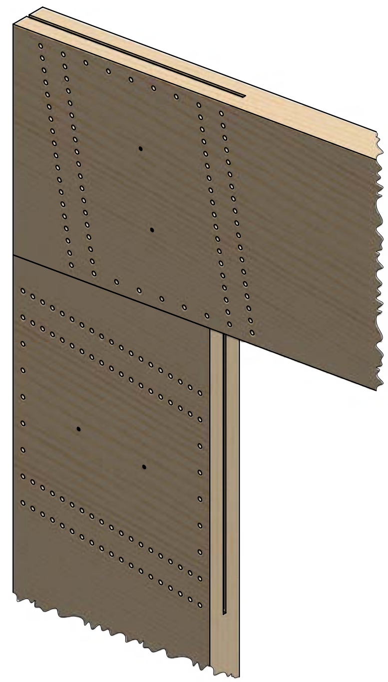
SECTION B-B



ELEVATION



SECTION A-A



NOTES-

- Predrill head diameter Ø11x15mm hole, to position dowel and prevent splitting near the head.
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- Use G250 steel plate to allow cutting heads to penetrate.
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- Ductility of connection μ=1.25-1.5 for seismic design.

MOMENT KNEE CONNECTION, WS DOWEL AND STEEL PLATE (MKCWSS - 600x90 LVL13)

NZ Wood For a better world

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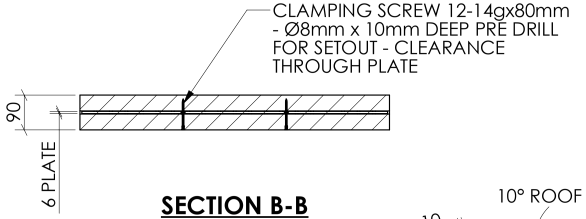
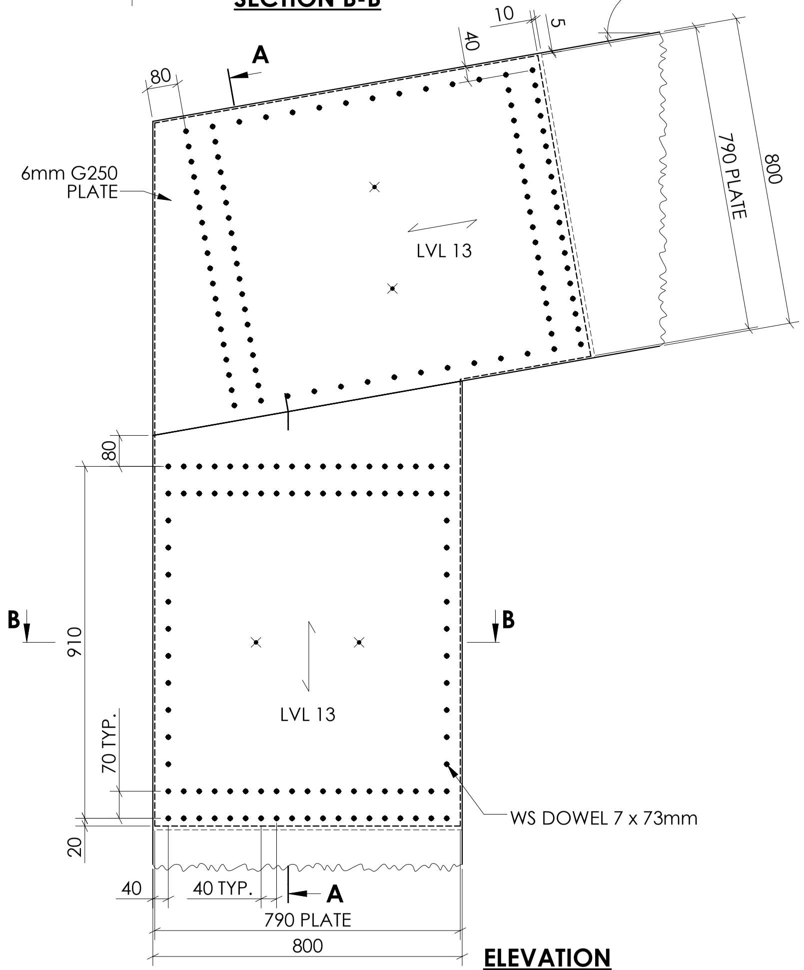
DRAWING NO.
MKCWSS - 600x90 LVL13

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	4/03/19
B	UPDATED	DJS	5/01/20
C	UPDATED	DJS	26/02/20
D	UPDATED	DJS	31/03/20

MATERIAL	MEMBER SIZE (mm)	G250 PLATE THICKNESS (mm)	G250 PLATE WIDTH (mm)	TOTAL NUMBER OF WS DOWELS TO EACH GROUP	MOMENT CAPACITY, (kNm) for K_1=1.0 DOWEL CAPACITY,ØM	PLATE YIELD CAPACITY, ØM	MEMBER CAPACITY, ØM
LVL 13	800 x 90	6	790	96	165	140	248



SECTION B-B



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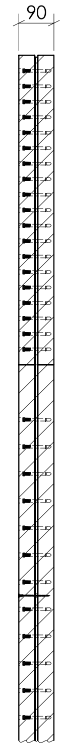
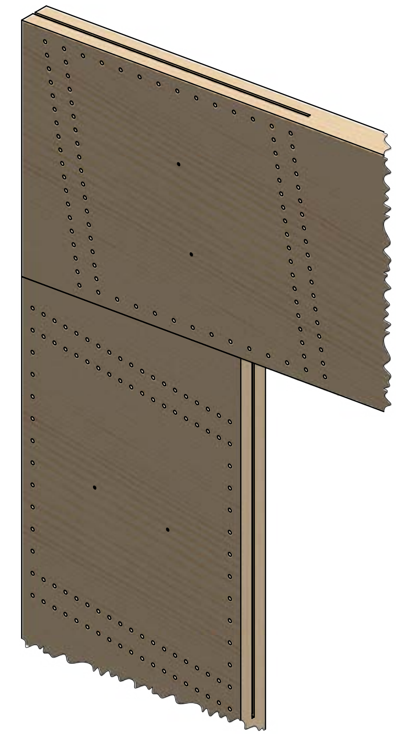


SECTION A-A



NOTES-

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- Use G250 steel plate to allow cutting heads to penetrate.
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- Ductility of connection μ=1.25-1.5 for seismic design.

MOMENT KNEE CONNECTION, WS DOWEL AND STEEL PLATE (MKCWSS - 800x90 LVL13)

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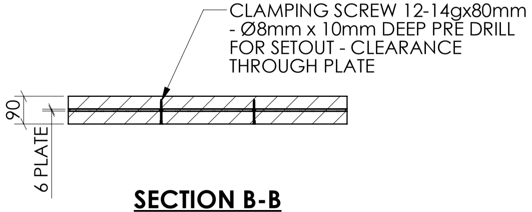
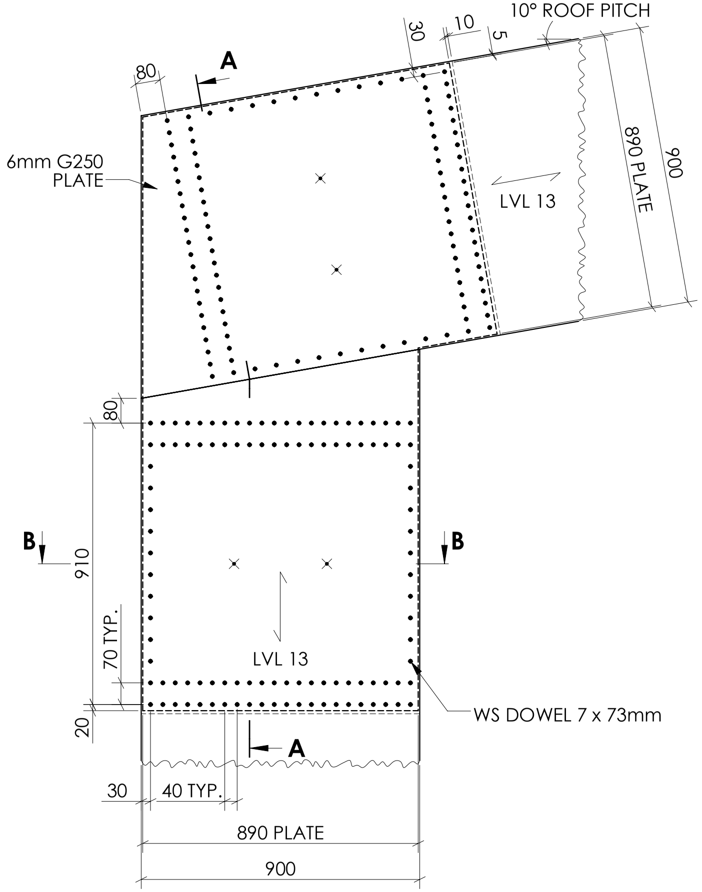
DRAWING NO.
MKCWSS - 800x90 LVL13

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	4/03/19
B	UPDATED	DJS	5/01/20
C	UPDATED	DJS	26/02/20

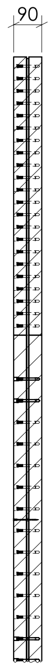
MATERIAL	MEMBER SIZE (mm)	G250 PLATE THICKNESS (mm)	G250 PLATE WIDTH (mm)	TOTAL NUMBER OF WS DOWELS TO EACH GROUP	MOMENT CAPACITY, (kNm) for K_1=1.0: DOWEL CAPACITY, ØM	MOMENT CAPACITY, (kNm) for K_1=1.0: PLATE YIELD CAPACITY, ØM	MOMENT CAPACITY, (kNm) for K_1=1.0: MEMBER CAPACITY, ØM
LVL 13	900 x 90	6	890	108	191	178	308



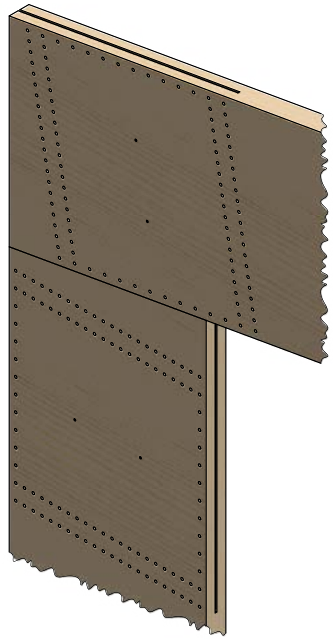
SECTION B-B



ELEVATION



SECTION A-A



NOTES-

- Predrill head diameter Ø11x15mm hole, to position dowel and prevent splitting near the head.
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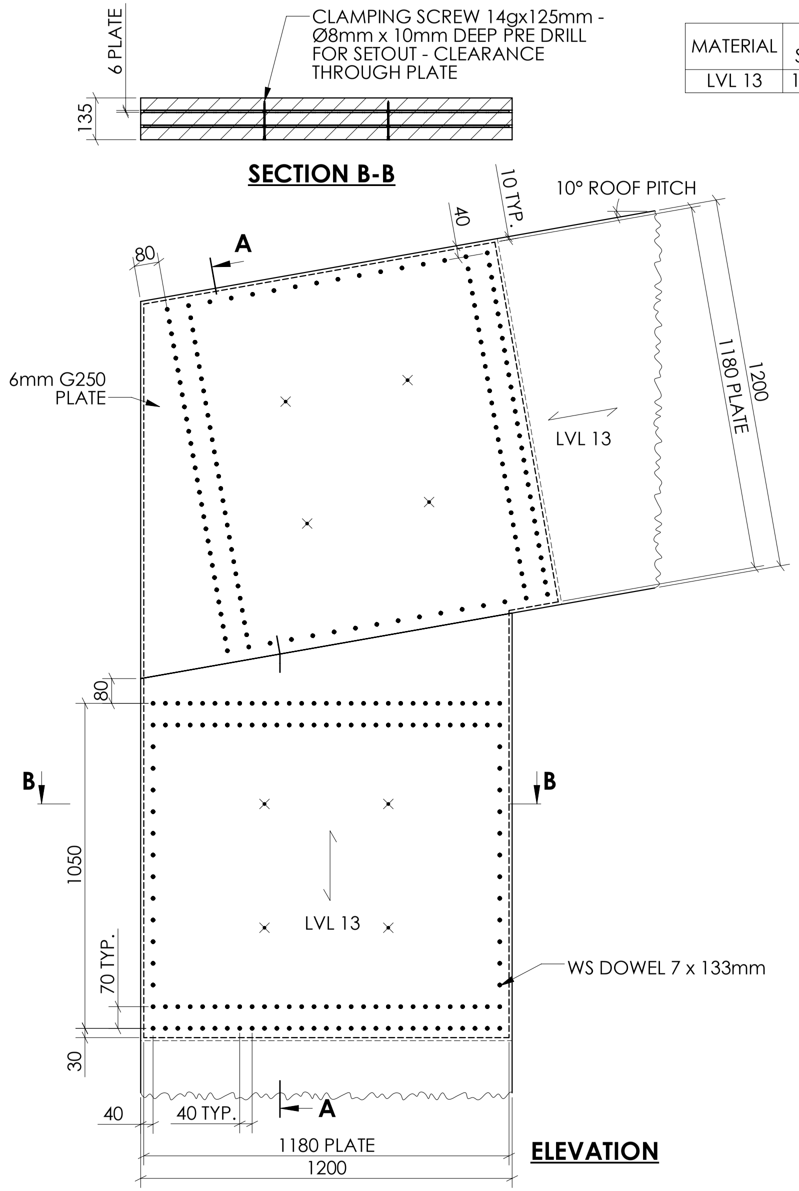
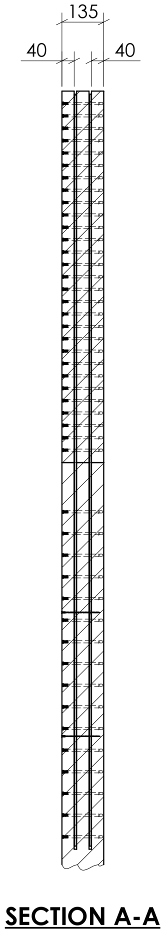
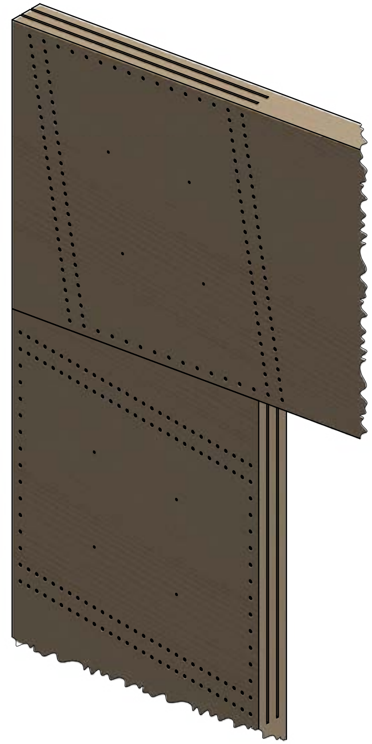
- Use G250 steel plate to allow cutting heads to penetrate.
- Capacities of the WS Dowels are taken from WS-01 (Rothoblaas) Technical data sheet using an LVL density of 480kg/m3.
- Ductility of connection μ=1.25-1.5 for seismic design.

MOMENT KNEE CONNECTION, WS DOWEL AND STEEL PLATE (MKCWSS - 900x90 LVL13)	NZ Wood For a better world — www.nzwood.co.nz design@nzwood.co.nz	DRAWING NO.: MKCWSS - 900x90 LVL13	REV	REVISION DESCRIPTION	DRN	DATE
			A	ISSUED FOR INFORMATION	DJS	4/03/19
			B	UPDATED	DJS	5/01/20
			C	UPDATED	DJS	27/02/20



MATERIAL	MEMBER SIZE (mm)	G250 PLATE THICKNESS (mm)	G250 PLATE WIDTH (mm)	TOTAL NUMBER OF WS DOWELS TO EACH GROUP	MOMENT CAPACITY, (kNm) for K_1=1.0 DOWEL CAPACITY,ØM	PLATE YIELD CAPACITY, ØM	MEMBER CAPACITY, ØM
LVL 13	1200 x 135	2/ 6	1180	140	731	627	757





NOTES-

- Predrill head diameter Ø11x15mm hole, to position dowel and prevent splitting near the head.
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- Use G250 steel plate to allow cutting heads to penetrate.
- Capacities of the WS Dowels are taken from WS-01 (Rothoblaas) Technical data sheet using an LVL density of 480kg/m3.
- Ductility of connection μ=1.25-1.5 for seismic design.

MOMENT KNEE CONNECTION, WS DOWEL AND STEEL PLATE (MKCWSS - 1200x135 LVL13)

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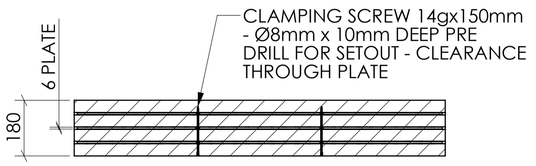
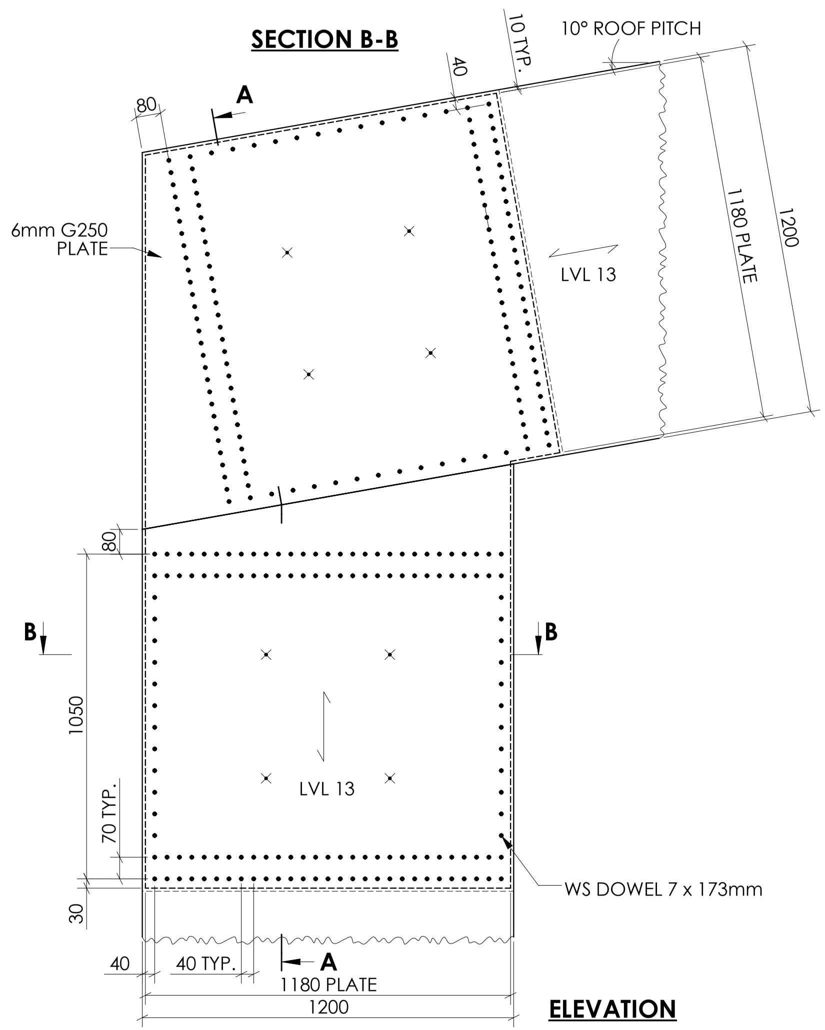
DRAWING NO.
MKCWSS - 1200x135 LVL13

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	4/03/19
B	UPDATED	DJS	5/01/20
C	UPDATED	DJS	27/02/20

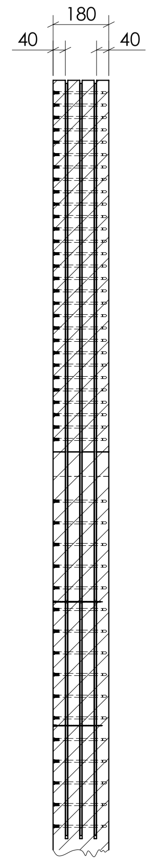


SECTION B-B

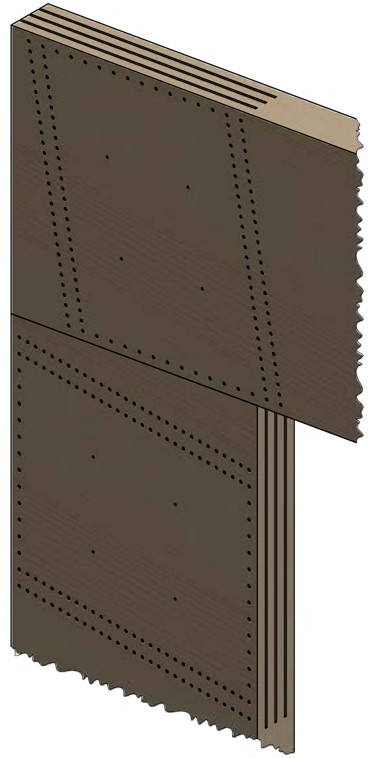
					MOMENT CAPACITY, (kNm) for K_1=1.0		
MATERIAL	MEMBER SIZE (mm)	G250 PLATE THICKNESS (mm)	G250 PLATE WIDTH (mm)	TOTAL NUMBER OF WS DOWELS TO EACH GROUP	DOWEL CAPACITY,ØM	PLATE YIELD CAPACITY, ØM	MEMBER CAPACITY, ØM
LVL 13	1200 x 180	3/6	1180	140	1006	940	993



ELEVATION



SECTION A-A



NOTES-

- Predrill steel plate (when positioned) with Ø6mm drill. Due to WS dowel drilling through 3 plates.
- Predrill head diameter Ø11x5mm hole, to position dowel and prevent splitting near the head.
- WS Dowels self drills into timber and steel. Use a 1500rpm driver.
- Use G250 steel plate to allow cutting heads to penetrate.
- Capacities of the WS Dowels are taken from WS-01 (Rothoblaas) Technical data sheet using an LVL density of 480kg/m3.
- Ductility of connection μ=1.25-1.5 for seismic design.

MOMENT KNEE CONNECTION, WS DOWEL AND STEEL PLATE (MKCWSS - 1200x180 LVL13)

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DRAWING NO.
MKCWSS - 1200x180 LVL13

REV	REVISION DESCRIPTION	DRN	DATE
A	ISSUED FOR INFORMATION	DJS	4/03/19
B	UPDATED	DJS	5/01/20
C	UPDATED	DJS	27/02/20
D	UPDATED	DJS	31/03/20

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